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NATIONAL DAM INSPECTION PROGRM. GUNTER VALLEY DAM (INVENTORY NU--ETC(U)

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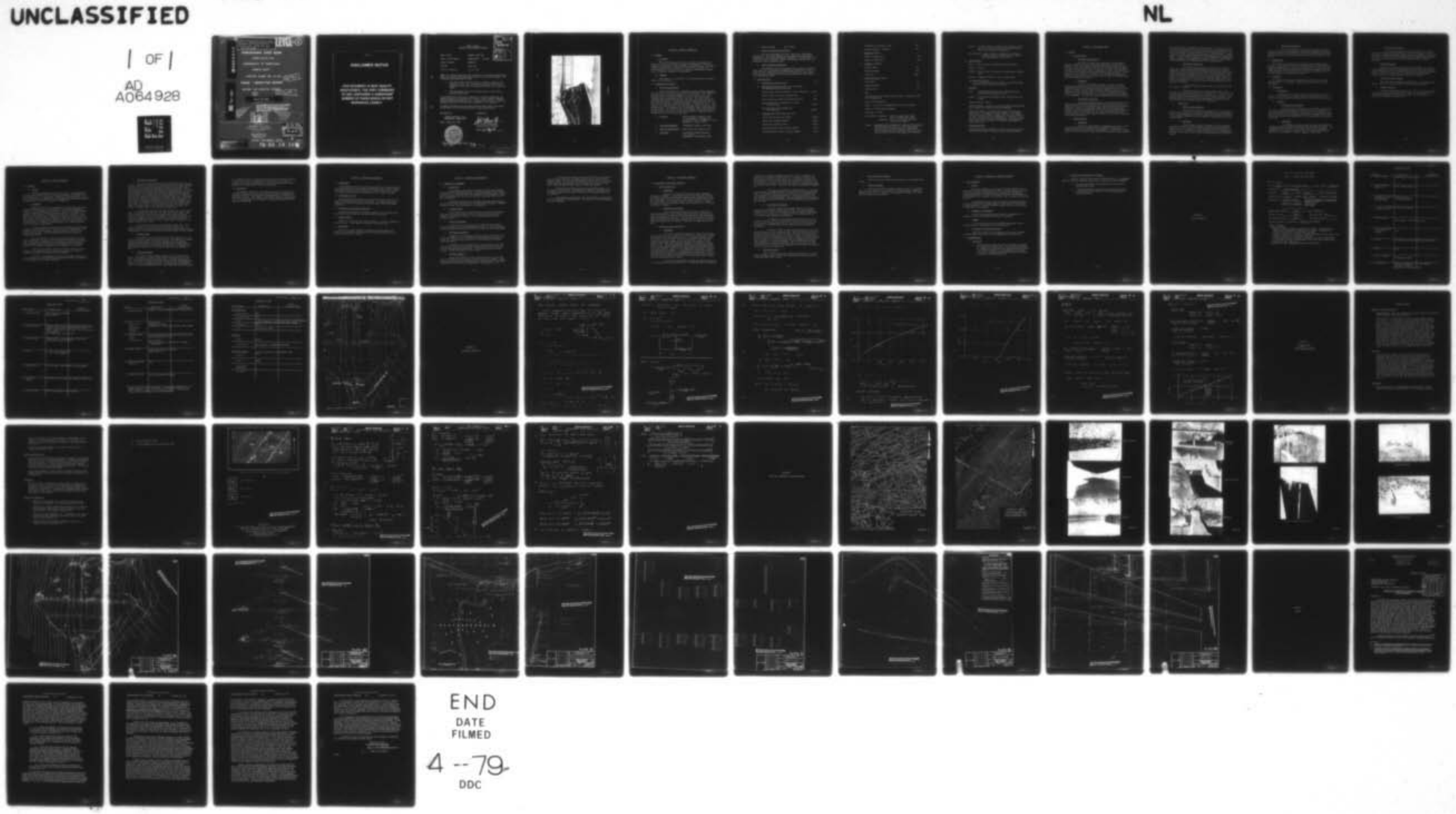

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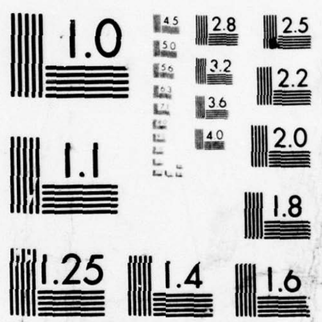


MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS-1963-A

SUSQUEHANNA RIVER BASIN

GUNTER VALLEY DAM

COMMONWEALTH OF PENNSYLVANIA

FRANKLIN COUNTY

INVENTORY NUMBER NDS PA-323

PHASE I INSPECTION REPORT

NATIONAL DAM INSPECTION PROGRAM

11 Aug 78

DACW31-78-C-0044

Prepared For
DEPARTMENT OF THE ARMY
Baltimore District, Corps of Engineers
Baltimore, Maryland

by

BERGER ASSOCIATES, INC
CONSULTING ENGINEERS
HARRISBURG, PA

AUGUST 1978

DISCLAIMER NOTICE

PHASE I REPORT
NATIONAL DAM INSPECTION PROGRAM

Name of Dam:	GUNTER VALLEY DAM
State & State Number:	PENNSYLVANIA - 28-102-A
County Located:	FRANKLIN
Stream:	Trout Run
Date of Inspection:	July 6, 1978

Based on a visual inspection, past performance and available engineering data, the dam and its appurtenances appear to be in good condition. The following recommendations are made:

1. The owner shall repair the weirs to operable condition and monitor the flow. If a quantitively increase would occur or turbidity in the water is discovered remedial action shall be taken.

2. The owner shall repair the spillway slab if further deterioration would occur.

In accordance with the Corps of Engineers' evaluation guidelines, the spillway capacity is inadequate for passing the PMF (Probable Maximum Flood) without overtopping the dam. However, this project is capable of passing 75 percent of the PMF and is considered to be adequate.

A formal surveillance and downstream warning system shall be developed by the owner to be used during periods of high precipitation.

SUBMITTED BY:

BERGER ASSOCIATES, INC.
HARRISBURG, PENNSYLVANIA

DATE: August 25, 1978

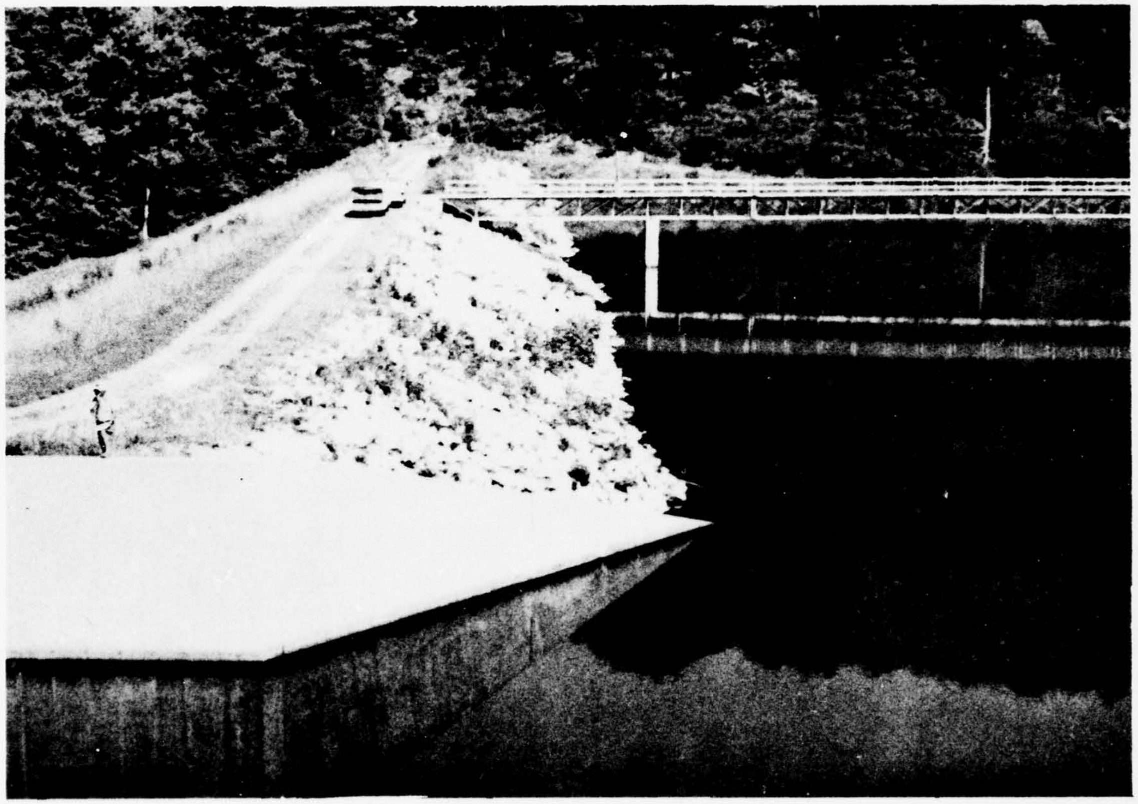
COMMONWEALTH OF PENNSYLVANIA
REGISTERED PROFESSIONAL
RODNEY VINCENT HOUSEAL
ENGINEER
No. 4756-E

Rodney V. Houseal

APPROVED BY:

John H. Kenworthy
JOHN H. KENWORTHY
LTC, Corps of Engineers
Acting District Engineer

DATE: 25 Aug 78



OVERVIEW

ABSTRACT

SECTION 1 - PROJECT INFORMATION

1.1 GENERAL

A. Authority

The Dam Inspection Act, Public Law 92-367, authorized the Secretary of the Army, through the Corps of Engineers, to initiate a program of inspections of dams throughout the United States. The Phase I Inspection and Report is limited to a review of available data, a visual inspection of the dam site and the basic calculations to determine the hydraulic adequacy of the spillway.

B. Purpose

The purpose is to determine if the dam constitutes a hazard to human life and property.

1.2 DESCRIPTION OF PROJECT

ABSTRACT

A. Dam and Appurtenances

Gunter Valley Dam is a homogeneous rolled earthfill embankment structure with a spillway located in the left abutment. For a general plan and typical section see Appendix D, Plates VII and VIII. The earthfill dam has a hydraulic height of 83 feet above the original streambed and an embankment length of 550 feet. The top of dam is at elevation 1028 and the spillway crest is at elevation 1015 and has a 73 feet long ogee section. The spillway channel is formed with a concrete slab and walls and has a length of 350 feet. At the end of the spillway channel is a stilling basin. The control/intake tower is located at the upstream side of the embankment and is accessible with a footbridge from the dam breast. A 4 feet by 5 feet cast-in-place conduit was used as a by-pass during construction and can now be used as a drawdown facility, controlled by a 30 x 24 inch sluice gate.

B.	Location:	Lurgan Township, Franklin County U.S. Quadrangle, Doylesburg, Pa. Latitude 40°-08.4', Longitude 77°-40.3' (Appendix D, Plates I & II)
C.	Size Classification:	Intermediate (Height is 83 feet)
D.	Hazard Classification:	Significant (See Section 3.1.E)
E.	Ownership:	Shippensburg Borough Authority P. O. Box 129 Shippensburg, Pennsylvania 17257

F. Purpose of Dam: Water Supply

G. Design and Construction History

The dam was designed by Glace & Glace, Inc., Harrisburg, Pennsylvania. The Permit Application was approved by Pennsylvania Department of Environmental Resources (PennDER) in October, 1960. The contractor was E. D. Plummers & Sons, Chambersburg, Pennsylvania and construction was completed in 1961.

H. Normal Operating Procedures

The reservoir has been constructed and is used for domestic water supply for the Borough of Shippensburg, Pennsylvania. Water is taken from the impounded lake at different elevations at the intake tower and carried through a 16-inch pipe located inside the conduit to the distribution system.

1.3 PERTINENT DATA

A. Drainage Area (square miles) 6.7

B. Discharge at Dam Site (Cubic feet per second)
See Appendix B for calculations

Maximum known flood at dam site - June, 1972 (Est.)	1,420
Outlet tunnel at low pool elevation 960	54
Outlet tunnel at normal pool elevation 1015.0	186
Spillway capacity at maximum design pool Elevation 1023	6,200
Spillway capacity at maximum pool Elevation 1028	12,820

C. Elevation (feet above mean sea level)

Top of dam (low point of camber)	1,028.0
Maximum pool design surcharge	1,023.0
Normal pool (spillway crest)	1,015.0
Upstream portal invert of outlet conduit	951.7
Downstream portal invert of outlet conduit	943.4

Streambed at centerline of dam	945
Maximum tailwater - Estimate	955

D. Reservoir (miles)

Length of maximum pool	0.8
Length of normal pool	0.6

E. Storage (acre-feet)

Spillway crest	640
Design surcharge	870
Top of dam	1,040

F. Reservoir Surface (acres)

Top of dam	37
Design surcharge	32
Spillway crest	26

G. Dam

For general plan and typical sections refer to Plates VII and VIII of Appendix D.

Type: Rolled earthfill.

Length: 550 feet of embankment and 74 feet of spillway.

Height: 83 feet above streambed.

Top Width: 20 feet.

Side Slopes: Upstream - 2.5H to 1V above Elev. 988.0
3.0H to 1V below Elev. 988.0
Downstream - 2.5H to 1V and a 10 feet wide berm at Elev. 988.0

Zoning: Homogeneous rolled earthfill of selected fill material. Upstream slope protected by a 2-foot thick dumped rock riprap on a 1-foot gravel bed. Downstream slope is seeded and has a dumped rockfill toe in the valley section where the dam height is the greatest.

Cutoff: A cutoff trench is located on the centerline of the dam. The trench was excavated to solid rock and filled with selected fill material.

Grout Curtain: A grout curtain is indicated on the longitudinal profile of dam and was to be 60 feet deep (See Appendix D, Plate IX).

H. Outlet Conduit

Type: 4 feet by 5 feet cast-in-place concrete conduit.

Length: 394 feet.

Closure: 30 by 24-inch sluice gate at upstream end on intake tower.

Access: Bridge from breast of dam to intake tower.

Regulating Facilities:
30 x 24 inch sluice gate on conduit. Two 24-inch slide gates on 16-inch pipe for water supply.

I. Spillway

Type: Uncontrolled ogee weir with training walls and concrete lined rectangular chute.

Length: 73 feet between abutment walls, including pier for bridge.

Crest elevation: 1015.0.

Upstream channel: Excavated to elevation 1009.0 and protected with a blanket of riprap and concrete wingwalls.

Downstream channel: The water flows over the 73 wide ogee section into a rectangular concrete channel. This channel tapers down from a width of 73 feet to 28 feet in a length of 130 feet, and widens further downstream to a width of 42 feet and ends in an 82 feet long stilling basin. The slope of the chute varies from 23 percent to 10 percent (see Appendix D, Plate XI and XII).

J. Regulating Outlets

The regulating outlet includes a low flow inlet to the outlet conduit with an invert elevation of 951.7 in intake tower.

SECTION 2 - ENGINEERING DATA

2.1 DESIGN

A. Data Available

1. Hydrology and Hydraulics

The files of Pennsylvania Department of Environmental Resources (PennDER) contained the Permit Application Report, dated October 7, 1960. This report stated that the spillway capacity requirement for this dam was 6030 cfs (C-Curve) and that the design discharge was 6200 cfs, which would leave a freeboard of 5 feet. The lake area is 26 acres and the storage is 208 million gallons (640 acre-feet). Calculations for the influence of the pier on the spillway discharge and hydraulic calculations by the engineer for the length of stilling basin were also available for review.

2. Embankment

The files of PennDER contained a full set of design drawings, which include a general plan, typical sections of the dam, longitudinal profile of dam with geological information and test boring results. Results of direct shear test and compaction tests on soil samples from test pits were also in the file. The engineer also submitted slip circle calculations for the upstream and downstream slopes of the embankment. All this data was reviewed and found to be adequate.

3. Appurtenant Structures

The design drawings include all structural details of the appurtenant structures and indicates under the general notes that the concrete used in the spillway walls had a 28-day compressive strength of 4,000 psi. Concrete in slabs and footers had a 28-day compressive strength of 2,500 psi and concrete for the intake tower was designed for concrete with a f'c of 5,000 psi. The files contained design calculations for the cast-in-place concrete conduit, but not for the spillway and stilling basin walls.

B. Design Features

1. Embankment

The dam is constructed as a homogeneous fill with a rock toe drain separated by a filter from the selected fill material. A blanket filter is not indicated on the drawings. A trench with a bottom width of 12 feet and 1H to 1V side slopes was excavated on the centerline

of dam through the overburden to "solid bedrock". The profile of the dam (Plate IX, Appendix D) indicates that a 60 feet deep grout curtain was to be used over the full length of the dam and spillway weir. A 2-foot thick dumped rock riprap is shown on the upstream slope. A safety factor of 1.6 was found for sudden drawndown condition on the upstream slope. A slope stability analysis for the downstream slope indicates a factor of safety of 1.9 under steady seepage condition. Cutoff walls were provided where the embankment meets the spillway wall.

2. Appurtenant Structures

The conduit was constructed in sections of 29.75 feet length with keyed construction and expansion joints, which include waterstops. Design calculations are in the files and cutoff walls were placed throughout the length of the conduit at 40 feet centers. The intake tower footer was placed on rock. The total height of the tower above the footing is 88 feet and the cross sections varies with the height. The top section is 2 feet by 1.5 feet and carries a 10.5 feet by 8.5 feet platform. All stems for gate controls are mounted on the outside of the tower.

A two-span footbridge gives access to the platform from the breast of the dam.

The spillway weir has a concrete cutoff wall on the upstream side extending a minimum of 6 feet into rock. The pier for the spillway bridge and the ogee section are monolythic. The walls for the weir abutments and the forebay walls are set on spread footings. The spillway chute and stilling basin are designed as a U-shape with the slab thickened near the walls (Plate XII, Appendix D).

C. Design Data

1. Hydrology and Hydraulics

The available design data consisted of a C-curve value of 6,030 cfs and a design Q of 6,200 cfs, which would leave a freeboard of 5 feet. Calculations were also made to check the influence of the pier on the spillway discharge and on the hydraulic length required for the stilling basin.

2. Embankment

Slope stability analysis was made for the upstream and downstream slopes for a sudden drawdown and steady seepage condition respectively. The weight of fill was assumed to be 131.5 lbs/cu.ft., weight of rock is 113 lbs/cu.ft. and a cohesion value of 1,000 lbs/sq.ft. was used. These values were obtained from laboratory test on test pit samples.

3. Appurtenant Structures

Plate XI, Appendix D, has the general structural notes, indicating type of reinforcement and compressive strength of concrete. Besides the analysis of the conduit, which was designed for dry fill and saturated fill, no other design criteria or data were available in the files.

2.2 CONSTRUCTION

The available construction data consisted of the design detail drawings. There were no records of as-built drawings. Available construction photographs were not in the microfiche for review. The visual inspection indicates that construction was done in accordance with the design drawings. The appearance of the structures indicated that the work was performed by a qualified contractor.

2.3 OPERATION

The purpose of the dam and appurtenant structures is to supply domestic water to the Borough. Formal records of operation are not maintained.

2.4 EVALUATION

A. Availability

A complete set of design drawings is available in the file of PennDER. These files also contain structural calculations for the conduit and some hydraulic calculations for the weir and stilling basin.

B. Adequacy

1. Hydrology and Hydraulics

Design criteria and data were not available for review in the files except that a design Q of 6,200 would leave a freeboard of 5 feet and exceeds the requirements of the C-curve. Area capacity curve, outlet works rating curve, spillway rating curve, frequency curve, unit hydrograph, design flood hydrograph or flood routings were not available for review.

2. Embankment

The embankment design was based on field and laboratory testing and two slope stability analyses were available for review. The embankment design is shown in the typical section and is generally considered to be adequate. However, there is no internal drainage or rock toe on the abutments to control seepage in these areas.

3. Appurtenant Structures

A review of the design drawings indicates that all structures were excellent detailed and appear to be well designed. Cutoff walls on the conduit and between spillway wall and embankment are detailed. Weep holes in slab and walls are detailed and the visual inspection did not detect any serious deterioration or unstable conditions of the structure.

C. Operating Records

While no formal operating records were available for review, it was reported that no major problems have occurred since this facility became operational in 1961. Maximum discharge over the spillway occurred during the tropical storm Agnes (1972) when the pool level reached elevation 1018±. No damage occurred to the dam or spillway.

D. Post Construction Changes

There have been no reported modifications to the original dam design. Three weirs were installed in 1963 to measure leakage.

E. Seismic Stability

The dam is located in Seismic Zone 1 and it is considered that the static stability with normal safety factors is sufficient to withstand minor earthquake induced dynamic forces. No calculations or studies have been made to confirm this.

SECTION 3 - VISUAL INSPECTION

3.1 FINDINGS

A. General

The general appearance of the dam is good. The appearance of the facilities indicate that the dam was constructed in accordance with the plans and that all appurtenant structures are well maintained. The visual checklist is in Appendix A. Photographs taken during the inspection are reproduced on Plates III through VI, Appendix D.

B. Embankment

Heavy weed growth on the downstream slope prevents very close inspection of this slope in the summertime. No signs of sloughage, erosion or slope movements were detected. Since the dam became operable in 1961, leakage has occurred at several locations. See Sketch 1 in Appendix A for locations of leakage points and weir locations. These weirs were installed in 1963. Seepage is coming out of the rock toe on both sides of the conduit outlet structure. The weir near the left wing of the outlet is not operable at present and flow is by-passing this measuring device. Seepage is also coming out of the rock toe about 50 feet to the right of the conduit.

A stream of water is running along the surface along the right wingwall of the spillway chute. The water was clear and although this stream has been present for many years, no gully or erosion of the surface has occurred. The weir measuring this flow was destroyed during Agnes in 1972.

The main concern for this water is the possibility of hydrostatic pressure on this wall, which was constructed without weep holes. Most of this water originates in an area where a natural ridge had not been disturbed during construction. Representatives of the Borough stated that the amount of leakage had not increased over the years.

The berm was wet over a length of 150 feet, just west of the spillway. The slope above the berm was dry and this wet condition appears to be caused by rainwater and poor drainage.

The top of the embankment was level and straight covered with stone and grass. The upstream slope has dumped rock with some growth on it, but appeared to be in good condition.

C. Appurtenant Structures

The intake structure was in good condition and has three gate operator stands; two for 24-inch sliding gates for water supply purposes and one for a 30 x 24 inch sluice gate to draw down the lake. This sluice gate has probably not been operated since construction and the representative of the Borough was hesitant to open the gate. The access to the intake tower is over a 173 feet long footbridge, which was in good condition. The conduit has an outlet structure on the downstream side with an impact baffle and was in excellent condition. The 16-inch pipe for water supply is hanging from the roof of the conduit over its full length. Some water was coming out of the conduit, although the sluice gate was supposed to be closed. Due to the low clearance inside the conduit, an attempt to find the source of this water could not be made. The source could be poor seating of the gate, a crack in the conduit or intake tower, or a leak in the 16-inch pipe.

The spillway, located in the left abutment was in good condition. The forebay walls and ogee weir were in excellent shape. Some minor cracking has occurred in the abutment walls under the bridge across the spillway. All joints appeared to be in good condition. The wall in the right spillway wall just below the bridge had deflected slightly (Appendix D, Plate V), but not significant in relation to its height (See Section 6.1.B.2).

All other walls were in good condition, except that it was noted that there were no weep holes in the right spillway wall. A slight spalling of the concrete spillway slab has occurred about 50 feet downstream of the spillway bridge and some reinforcement has been exposed.

D. Reservoir Area

The reservoir area is wooded and all banks appear to be clean and no indication of bank erosion was noticed. The watershed is located between the Blue Mountain and Kittany Mountain and is all wooded. During the construction of the second turnpike tunnel through these mountains, some siltation occurred in the reservoir. The Borough owns approximately 3,800 acres of the watershed and selective logging is done.

E. Downstream Channel

The conduit outlet joins the creek just below the stilling basin. The creek is a typical wooded valley mountain stream for the first mile and one-half and then passes under a bridge carrying Route 641. Just below the bridge, Trout Run confluences with the Conodoguinet Creek. There are no residences between the dam and the Conodoguinet. Roxbury is located approximately half a mile downstream of the confluence.

It is considered that the additional loss of life which would occur due to dam failure after overtopping would be limited to a few and the economic loss would be appreciable. The hazard classification is considered to be "Significant" for Gunter Valley Dam.

3.2 EVALUATION

The observed condition of the facility was good. Although considerable leakage exists, all water was clear and could originate from springs and natural flow through the embankment which is collected in the large toe drain. The leakage in the conduit should be investigated and it is recommended that the drawdown sluice gate be operated on at least an annual basis.

SECTION 4 - OPERATIONAL PROCEDURES

4.1 PROCEDURES

This impoundment dam was constructed to serve as a storage reservoir for drinking water for the Borough of Shippensburg and is one of several water supply facilities for the Borough. Water is taken from the lake as demands require through the 16-inch pipe in the intake structure.

4.2 MAINTENANCE OF DAM

The appurtenant structures are in excellent condition and have not required much maintenance since construction. The growth on the downstream slope should be controlled to prevent major future maintenance problems.

4.3 MAINTENANCE OF OPERATING FACILITIES

The water supply gates are operated regularly, but the sluice gate to draw down the lake has not been used since construction.

4.4 WARNING SYSTEM

There is no formal warning system in effect. The dam is checked on a weekly basis. A staff gage is located on the intake tower.

4.5 EVALUATION

The dam is in good condition, although little maintenance has occurred. The drawdown sluice gate has not been used and there is no formal warning system in effect.

SECTION 5 - HYDROLOGY AND HYDRAULICS

5.1 EVALUATION OF FEATURES

A. Design Data

The hydrologic and hydraulic analyses available from PennDER for Gunter Valley Dam indicated that no design hydrograph, flood routing, storage curve or discharge curves were submitted by the designer. There was a statement in the file that the spillway could pass 6200 cfs with 5 feet of freeboard.

A spillway rating curve and storage curve have been developed for this report using information in the construction drawings. Hydraulic computations made for this report are in Appendix B.

B. Experience Data

In the period since the dam has been constructed the maximum flood was that which occurred in 1972, for which the flow over the spillway is estimated at 1420 cfs. The spillway passed that flood without distress.

C. Visual Observations

On the date of the inspection, no conditions were observed that would indicate that the appurtenant structures of the dam could not operate satisfactorily during a flood event, until the dam is overtopped.

D. Overtopping Potential

Comparison of the estimated Probable Maximum Flood (PMF) peak inflow of 18,100 cfs with the estimated ultimate spillway capacity of 12,820 cfs, indicates that a potential for overtopping of the Gunter Valley Dam exists.

An estimate of the storage effect of the reservoir shows that this dam does not have the necessary storage available to pass the PMF without overtopping (see Appendix B). The spillway-reservoir system can pass a flood event equal to 75% of a PMF.

E. Spillway Adequacy

Gunter Valley Dam has a total storage capacity of about 1040 acre-feet and the overall height is 83 feet above the streambed. These dimensions indicate a size classification of "Intermediate". The hazard classification for this dam is "Significant" (see Section 3.1.E).

The recommended Spillway Design Flood (SDF) for a dam having the above classifications is between one-half and one PMF (Probable Maximum Flood). For this dam the PMF peak inflow is 18,100 cfs and the maximum spillway capacity with the water level at the top of the dam (Elev. 1028) is about 12,820 or 70% of the PMF peak inflow.

Although the spillway cannot pass the PMF peak inflow, it is considered to be adequate. Calculations in Appendix B indicates that the bridge superstructure does not influence the discharge capacity of the spillway.

The hydrologic analysis for this investigation was based upon existing conditions of the watershed. The effects of future development were not considered.

SECTION 6 - STRUCTURAL STABILITY

6.1 EVALUATION OF STRUCTURAL STABILITY

A. Visual Observation

1. Embankment

There were no visual indications of undue embankment stresses or sloughage. The embankment was in generally good condition. The seepage from the toe drain was considered to be indicating a properly functioning toe drain; however, past records indicate large variation in seepage. The available records are over too short a period (3 months in 1963) to conclude if pool level, rainfall, etc., influences the amount of seepage.

2. Appurtenant Structures

Visual observations indicate no present stability or stress problems in any of the appurtenant structures. Some deterioration of the spillway chute slab has occurred and one wall has slightly deflected (Appendix D, Plate V). Some concern exists about the hydrostatic pressure behind the right spillway over the length of wall where water was running over the surface. There were no weep holes in this wall.

B. Design and Construction Data

1. Embankment

The files of PennDER contained the results of test pits and test results on soil samples from these pits. These tests included direct shear tests and compaction tests. The design drawings include the results of test borings. Based on the results of this testing program, the designer submitted a slope stability analysis for the upstream slope under sudden drawdown condition and reported a safety factor of 1.6. The slope stability analysis for the downstream slope under a steady seepage condition was also analyzed and a safety factor of 1.9 was reported. Based on this information, the design of the dam is considered to be adequate. It is noted that there is no internal drainage and no toe drain in the embankment at the abutments. However, no wet spots were noted on the embankment which is most likely due to the fact that the foundation material is more pervious than the embankment and is functioning as a drain.

In June 1962, considerable leakage was noticed at the toe of the dam and along the spillway wall. A report was made by D'Appolinia

Associates, Pittsburgh, Pennsylvania and this report recommended the installation of weirs to measure the flow. No actual sources of the leakage were discussed in the report, but the different possibilities were mentioned. The Rose Hill Sandstone outcrops in the reservoir and the fractures in this rock and its contact surfaces with adjacent shales could easily carry water. A copy of this report is included in Appendix E.

The weirs were installed in 1963 and records of readings were submitted to PennDER; however, no record of the weir location was found. The largest flow was read on Weir No.1 and is presumably the weir installed in the downstream channel. Quantities varied in 1963 from 356,000 to 76,000 gallons/day. The weir readings were discontinued and the weirs were made inoperable during the Agnes storm and have not been repaired.

2. Appurtenant Structures

A review of the design drawings indicates a properly engineered intake tower, conduit and spillway. Reinforcing appears adequate and a review of the foundations indicates correct assumptions of rock or soil foundation. The detailing of the spillway weir and chute applied good engineering techniques.

In Appendix C calculations have been made to check two points of concern. The first calculations checks the stresses in the spillway wall, assuming saturated fill to one foot below top of wall. An overstress of approximately 6% was found and this is considered acceptable.

On Sheets 3 and 4 a deflection calculation for the spillway wall adjacent to the bridge was made. This 28.5 foot high wall has deflected approximately one inch and this was reported first in 1965. Assuming that the footing does not rotate, an active soil pressure of 35 lbs/cu.ft. causes a deflection of 5/16-inch. However, the soil pressure will be higher due to some saturation and the U-shaped design makes a small rotation of the footing feasible. Above calculations are short term deflections. Long term deflection will be about twice as high. The deflection of the wall is not considered to be serious and a logical result from the type construction.

C. Operating Records

While no formal operating records were available, Mr. Smith, Borough Manager, stated that no major problems have occurred since the dam became operational in 1961.

D. Post Construction Changes

There have been no reported modifications to the original dam design.

E. Seismic Stability

This dam is located in Seismic Zone No.1 and it is considered that the static stability is sufficient to withstand minor earthquake induced dynamic forces. However, no calculations, studies, etc., were made to confirm this conclusion.

SECTION 7 - ASSESSMENT & REMEDIAL MEASURES

7.1 DAM ASSESSMENT

A. Safety

The visual inspection, the review of design drawings and the operational history indicates that the dam is in good condition and that it has been designed and constructed in accordance with acceptable engineering practice. Persistent leakage is occurring and although is not considered to be a hazard at the present time, it should be monitored closely.

In accordance with the Corps of Engineers' evaluation guidelines, the spillway and storage capacity of this project is sufficient to pass 75 percent of the PMF (Probable Maximum Flood) and the spillway is considered to be adequate.

B. Adequacy of Information

The available information for review is considered to be adequate to make a reasonable assessment of the project.

C. Urgency

It is considered that the recommended suggestions in this section should be implemented as soon as practical.

D. Necessity for Additional Studies

Additional studies are not required at this time. However, attention should be given to the recommendations presented below.

7.2 RECOMMENDATIONS

A. Facilities

1. The owner should repair the weirs and monitor the amount of leakage on a regular basis. This information should be correlated with previous weir readings and pool levels. If a change in quantity or any turbidity in the water would occur, immediate action should be taken to prevent a hazard to the downstream area.

B. Operation and Maintenance Procedures

Although the dam is maintained in good condition, it is considered important that the following items be given attention as soon as possible.

1. The owner shall repair the spillway slab if any further deterioration occurs.

2. A formal surveillance and downstream warning system shall be established by the owner to be used during periods of high precipitation.

APPENDIX A

VISUAL INSPECTION

CHECK LIST - DAM INSPECTION PROGRAM

PHASE I - VISUAL INSPECTION REPORT

NAD NO. 323

PA. ID # 28-102 NAME OF DAM Gunter Valley HAZARD CATEGORY Significant

TYPE OF DAM: Rolled Earthfill

LOCATION: Lurgan TOWNSHIP Franklin COUNTY, PENNSYLVANIA

INSPECTION DATE 7-6-78 WEATHER Clear - Sunny TEMPERATURE 70 - 80

INSPECTORS: H. Jongsma, R. Houseal Representing Borough of Shippensburg
Walter K.. Smith

A. Bartlett, R. Shireman Harold Myers
Earl Fickes

NORMAL POOL ELEVATION: 1015.0 AT TIME OF INSPECTION:

BREAST ELEVATION: 1028.0 POOL ELEVATION: 1015.1

SPILLWAY ELEVATION: 1015.0 TAILWATER ELEVATION: ______

MAXIMUM RECORDED POOL ELEVATION: Spillway + 30" (Agnes)

GENERAL COMMENTS:

Files - Drawings

D'Appolnia - Report suggests seepage due to springs. Weirs washed out during Agnes. Siltation at upper end was cleared. 36-3800 acres timbers in watershed. Systematically harvested.

Intake at 2 levels + one at bottom - blowoff take from top first valve first. Seepage was reduced or eliminated after Agnes.

Siltation problem due to turnpike construction. Low flow during summer. No residence below dam to Conodoguinet Creek (one cabin (summer home).

Attendance at dam - once a day.

Roxbury is on the Conodoguinet Creek below Trout Creek.

VISUAL INSPECTION

EMBANKMENT	OBSERVATIONS	REMARKS & RECOMMENDATIONS
A. SURFACE CRACKS	None evident on top of dam	
B. UNUSUAL MOVEMENT BEYOND TOE	None - dumped rock toe	
C. SLOUGHING OR EROSION OF EMBANKMENT OR ABUTMENT SLOPES	Weed cover heavy on downstream slope	
D. VERTICAL & HORIZONTAL ALIGNMENT OF CREST	No distress observed	
E. RIPRAP FAILURES	None evident - rock is dumped rock.	
F. JUNCTION EMBANKMENT & ABUTMENT OR SPILLWAY	Good	
G. SEEPAGE	Discharging along right spillway channel wall.	
H. DRAINS	None	
J. GAGES & RECORDER	Staff gage on tower.	
K. COVER(GROWTH)	Upstream - dumped rock with thorn growth. Top stone and grass. Downstream - heavy weed growth	

VISUAL INSPECTION

OUTLET WORKS	OBSERVATIONS	REMARKS & RECOMMENDATIONS
A. INTAKE STRUCTURE	Small tower with platform. Good condition	
B. OUTLET STRUCTURE	Concrete impact basin. Steady discharge from outlet. Seepage from rock toe on right side of outlet. Water - steady flow at left wing. Weir in this area - not operable. Flow from dumped rock toe drain.	
C. OUTLET CHANNEL	Grassed slopes - stone bottom to forest area then trees to edge of stream.	
D. GATES	2 - 24" on 16" water supply 1 - 30" x 24" drawdown.	
E. EMERGENCY GATE	24" x 30" gate. Gate seldom, if ever, opened.	
F. OPERATION & CONTROL	One water line from dam - 16" diameter reduces to 12"	
G. BRIDGE (ACCESS)	Steel truss bridge to tower platform.	

DAM NO. NAD 323

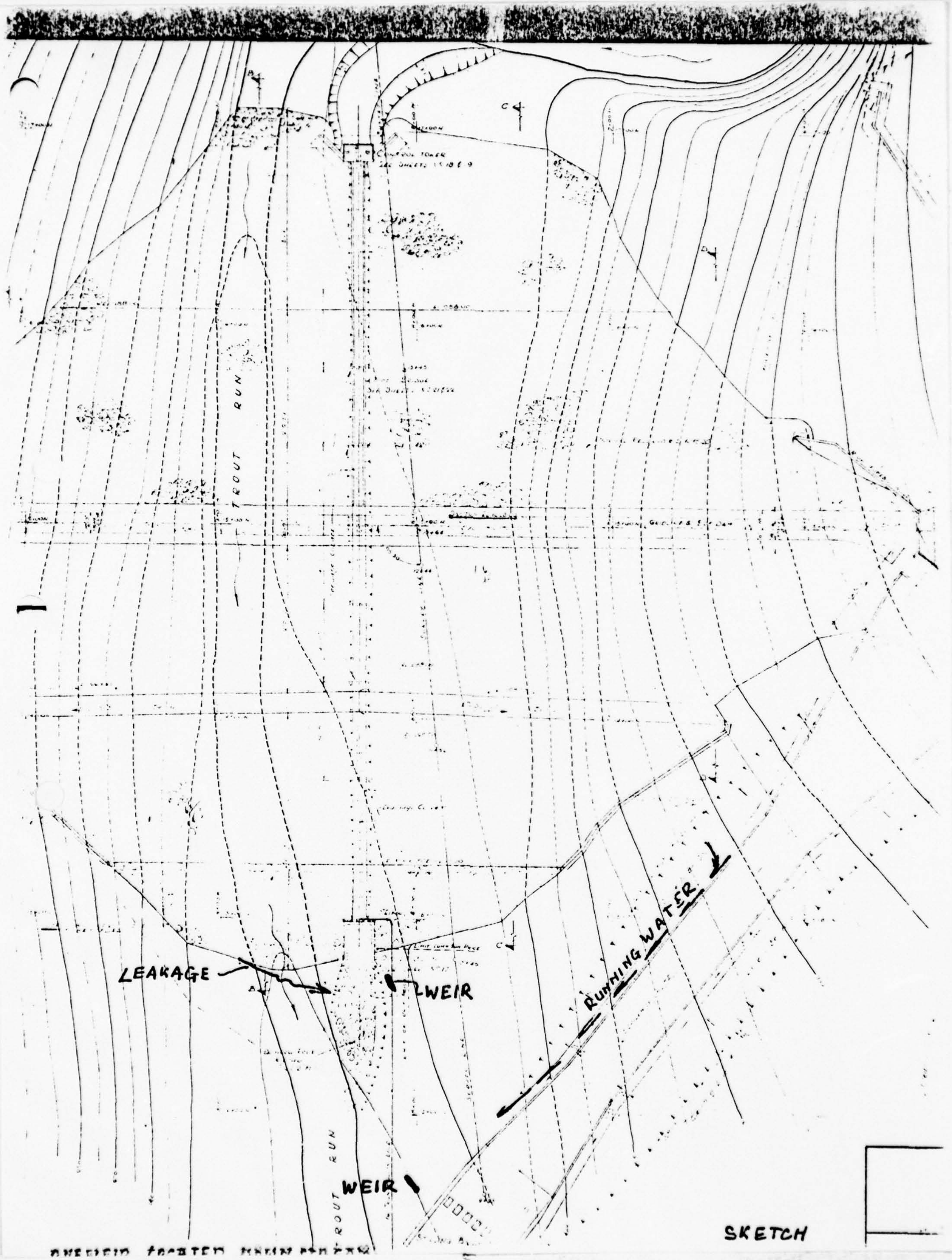
VISUAL INSPECTION

SPILLWAY	OBSERVATIONS	REMARKS & RECOMMENDATIONS
A. APPROACH CHANNEL	Forebay - concrete walls leading to ogee section.	
B. WEIR: Crest Condition Cracks Deterioration Foundation Abutments	Ogee Section Excellent condition. Cracking nil - slight cracks in wall under bridge.	
C. DISCHARGE CHANNEL Lining Cracks Spilling Basin	Slight displacement in vertical walls below ogee. Zero at bottom 2" at top. Good Condition. Some reinforcing bar sticking out of channel bottom slab.	
D. BRIDGE & PIERS	Bridge over spillway (concrete) Good condition	
E. GATES & OPERATION EQUIPMENT	None	
F. CONTROL & HISTORY	30" over spillway 1972 Agnes	

Seepage behind the right channel wall - considerable steady flow at surface adjacent to wall - no weep holes in the wall. Point is about midway between the spillway and the stilling basin.

VISUAL INSPECTION

MISCELLANEOUS	OBSERVATIONS	REMARKS & RECOMMENDATIONS
INSTRUMENTATION		
Monumentation	None	
Observation Wells	None	
Weirs	1 left of outlet, 1 Rt. spillway wall, 1 in main channel None are operating, could be used with repairs	
Piezometers	None	
Other	Staff gage on tower.	
RESERVOIR		
Slopes	Forest	
Sedimentation	Some problems - turnpike construction	
DOWNSTREAM CHANNEL	1.6 miles below dam is Conodoguinet Creek	
Condition	Good	
Slopes	Wooded	
Approximate Population	1	
No. Homes	1	

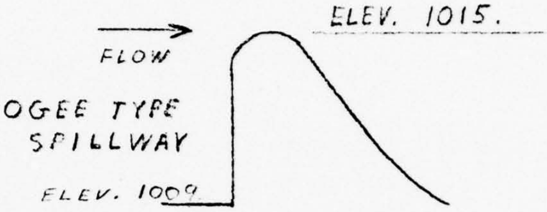

TROUT RUN
LEAKAGE
WEIR
RUNNING WATER
WEIR
TROUT RUN
SKETCH

APPENDIX B

HYDROLOGY/HYDRAULICS

MAXIMUM KNOWN FLOOD AT DAMSITE

BORO MANAGER INDICATED THAT THE MAXIMUM KNOWN FLOOD SINCE CONSTRUCTION OF THE DAM OCCURRED IN 1972. THE HEAD ON THE WEIR AT THAT TIME WAS ABOUT 3 FEET.



$L = 73' - 1' = 72'$
$H = 3'$
$C = 3.50$

$C = 3.80$ (DESIGN OF SMALL DAMS FIG. 249)

$Q = C L H^{3/2}$
$= 3.8 \times 72 \times (3)^{3/2}$

$= 1422$ SAY 1420 CFS

OUTLET WORKS

SLUICEWAY IS 4' x 5' CONCRETE TUNNEL. CONTROLLED BY 30" x 24" SLUICE GATE.

DISCHARGE THROUGH ORIFICE WITH POOL ELEV AT SPILLWAY CREST ELEV. TAILWATER ELEV = 955

$H = 1015 - 955 = 60'$

$A = \frac{30}{12} \times \frac{24}{12} = 5$ S.F.

$C_D = 0.6$

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$Q = C_D A \sqrt{2gH} = 0.6 \times 5 \times (2 \times 32.2 \times 60)^{.5} = 186$ CFS

AT LOW POOL $H = 960 - 955 = 5'$

$Q = C_D A \sqrt{2gH} = 0.6 \times 5 \times (2 \times 32.2 \times 5)^{.5} = 54$ CFS

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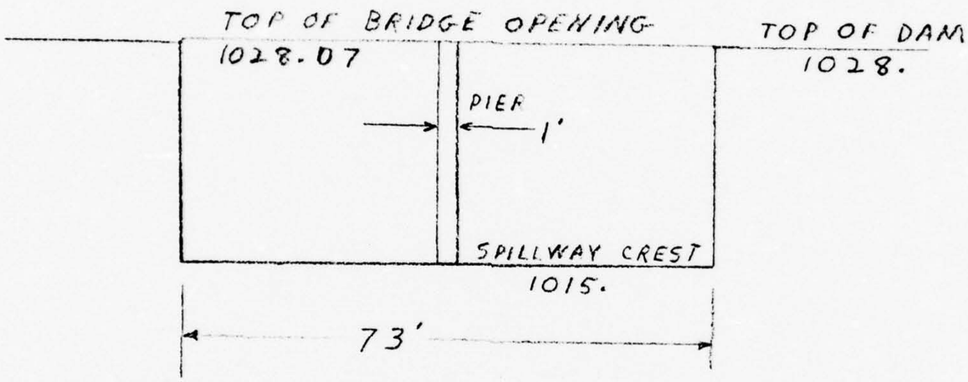
SUBJECT GUNTERS VALLEY DAM

SPILLWAY DISCHARGE WITH POOL ELEV. AT 1028
(TOP OF DAM)

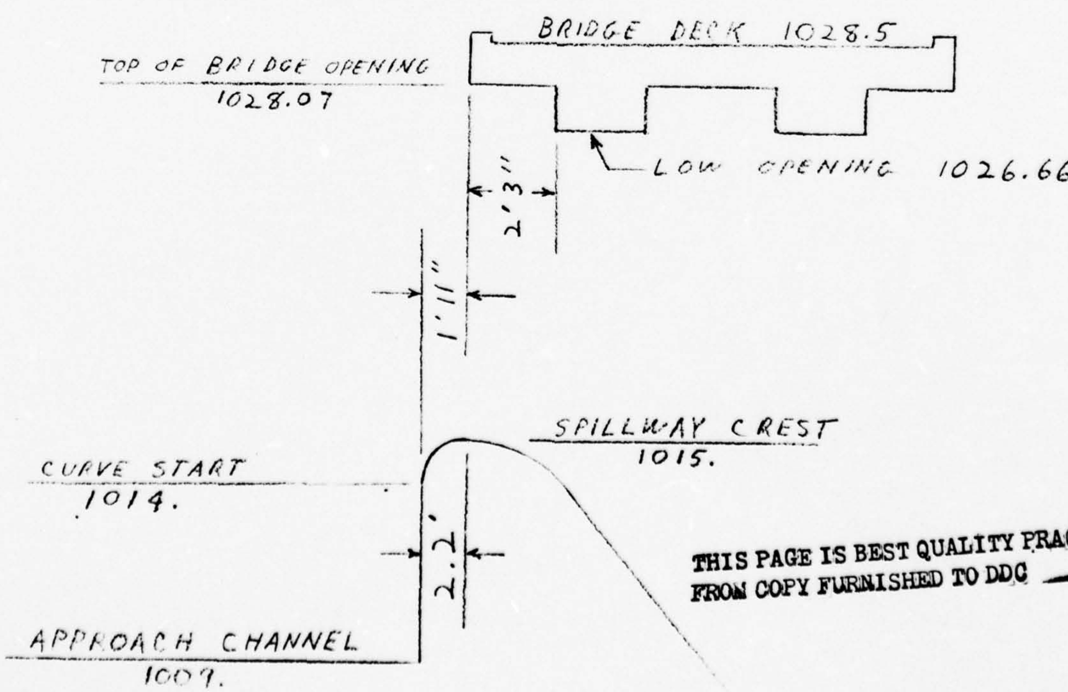
$H = 1028 - 1015 = 13'$

$Q = C L H^{3/2}$
$= 3.8 \times 72 \times (13)^{3/2}$

$= 12824$ USE 12820 CFS



BRIDGE OPENING



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SUBJECT GUNTERS VALLEY DAM

WATER ELEV. UNDER LOW OPENING $Q = 12820$ CFS

$X_B = 1'11'' + 2'3'' = 4'2''$

APPROACH VELOCITY $= 12820/(73 \times 19) = 9.24$ FPS
$H_V = 1.33$

TOTAL SPILLWAY HEAD $= 13 + 1.33 = 14.33' = H$

NAPPE DIMENSIONS: (EQN. 6-39, MORRIS + WIGGERT APPLIED HYDRAULICS)

$$\frac{Y_B}{H} = \left[0.15 - 0.45\left(\frac{H_V}{H}\right)\right] + \left[0.411 - 1.603\left(\frac{H_V}{H}\right) - \sqrt{1.568\left(\frac{H_V}{H}\right)^2 - 0.892\left(\frac{H_V}{H}\right) + 0.127}\,\right]\left(\frac{X_B}{H}\right) - \left[0.425 - 0.25\left(\frac{H_V}{H}\right)\right]\left(\frac{X_B}{H}\right)^2$$

$= .1082 + .0064 - .034 = .0806$

$Y_B = .0806 \times 14.33 = 1.16$

$$\frac{T}{H} = 0.57 - 2\left[\frac{H_V}{H} - 0.208\right]^2 e^{10\left(\frac{H_V}{H} - 0.208\right)}$$ (EQN 6-40 MORRIS + WIGGERT)

$= .5616$

$T = .5616 \times 14.33 = 8.05$

$T + Y_B = 8.05 + 1.16 = 9.21$

ELEV $= 9.21 + 1014 = 1023.21$

NOT AFFECTED BY BRIDGE

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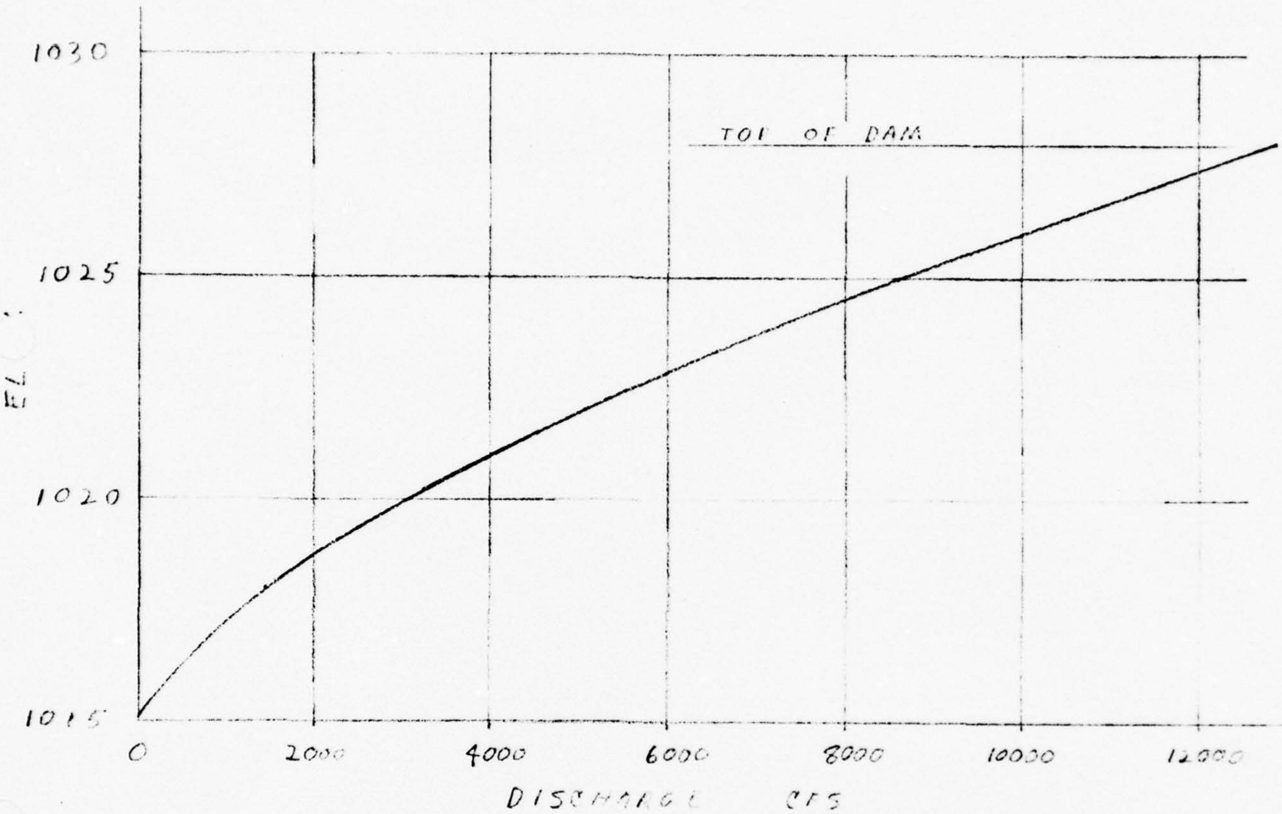
SUBJECT GUNTERS VALLEY DAM

SPILLWAY DISCHARGE CAPACITY



SIZE CLASSIFICATION

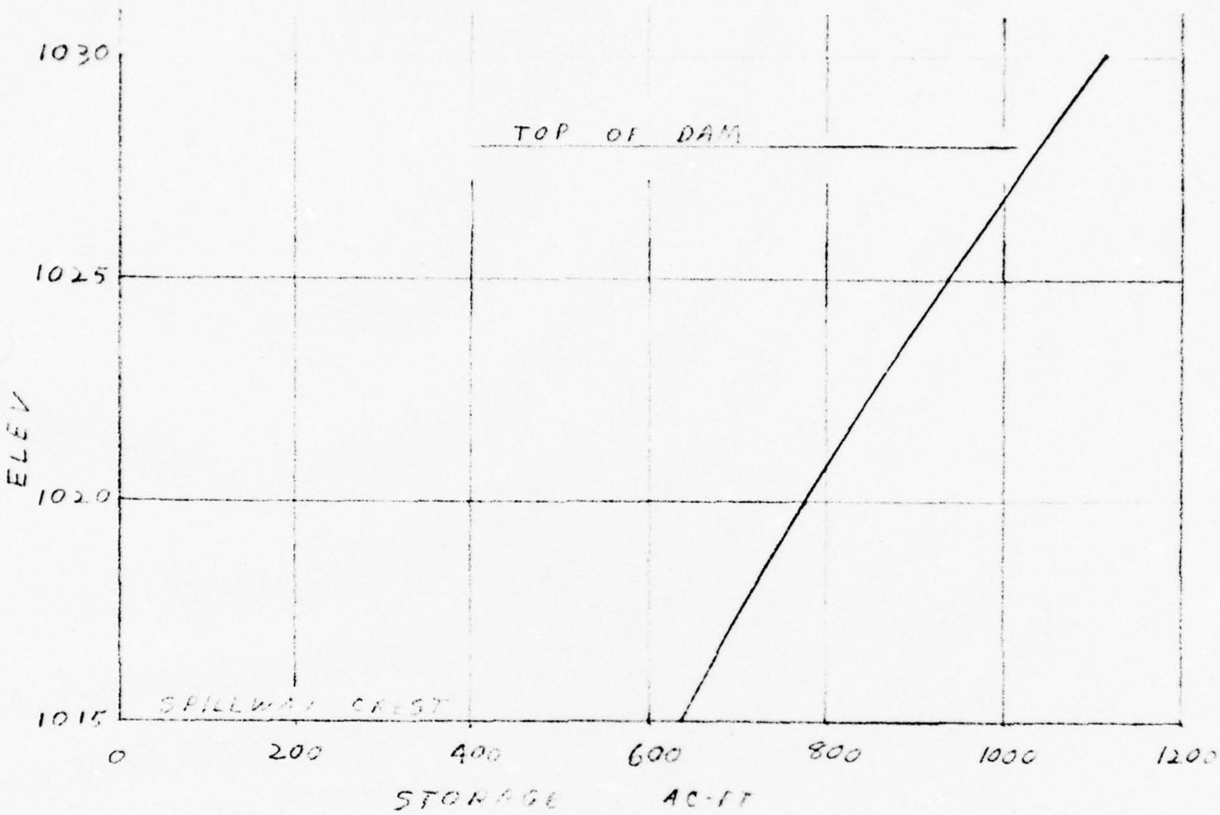
MAXIMUM STORAGE = 1040 ACRE-FT
HEIGHT = 1028 - 950 = 78'
SIZE CLASSIFICATION IS INTERMEDIATE

HAZARD POTENTIAL

NO HOUSES IN FIRST TWO MILES DOWNSTREAM.
BORO OF ROXBURY IS ABOUT FOUR MILES DOWNSTREAM
USE SIGNIFICANT.

BY FLS DATE 7/14/78 **BERGER ASSOCIATES** SHEET NO. 5 OF
CHKD. BY DR DATE 7/17/78 PROJECT
SUBJECT GUNNERS VALLEY DAM

STORAGE CAPACITY



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SUBJECT GUNTERS VALLEY DAM

PMF

DRAINAGE AREA = 6.7 SQ. MI.

PMF = 2700 CSM
T = 26 HR
} FROM CURVES FURNISHED BY CORPS OF ENGINEERS, BALT. DIST.

PMF = 2700 x 6.7 = 18090 USE 18100 CFS

VOL OF INFLOW = $18100 \times \frac{26}{24} \times .5$ = 9804 CFS - DAY
= 19412 AC-FT
= 54.4 INCHES RUNOFF (TOO HIGH)

USE 26 INCHES RUNOFF

26 x 6.7 x 53.3 = 9285 AC-FT

$\frac{\text{MAX. SPILLWAY DISCHARGE}}{\text{PEAK INFLOW}} = \frac{12820}{18100} = 0.708$ SAY 70%

$\frac{\text{REQD. RES. STORAGE}}{\text{VOL. OF INFLOW}} = 0.292$ FROM CORPS OF ENGINEERS SHORTCUT METHOD

REQD. RES. STORAGE = .3 x 9285 = 2711 AC-FT

STORAGE AVAILABLE BETWEEN ELEV 1015 AND 1028.

1040 - 638 = 402 AC-FT

402 < 2711

∴ DAM WILL BE OVERTOPPED

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SUBJECT GUNTERS VALLEY DAM

PROJECT CAPACITY

75% PMF

18100 x .75 = 13575 CFS
9285 x .75 = 6964 AC-FT

$$\frac{\text{MAX. SPILLWAY DISCHARGE}}{\text{75\% PMF PEAK INFLOW}} = \frac{12820}{13575} = .944 \quad \text{SAY } 94\%$$

$$\frac{\text{REQD. RES. STORAGE}}{\text{VOL OF INFLOW}} = 0.056$$

REQD. RES. STORAGE = .056 x 6964 = 390 AC-FT

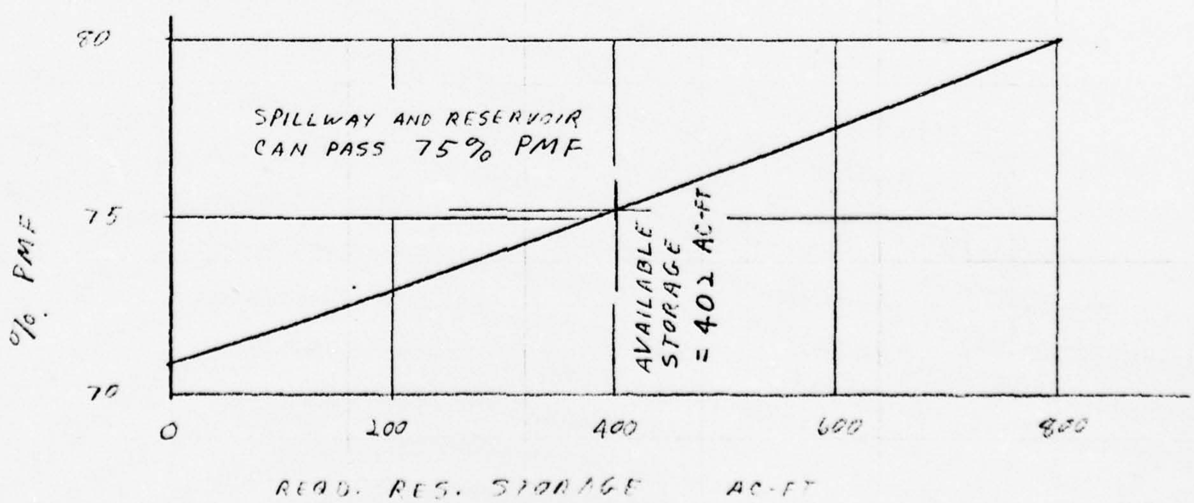
80% PMF

18100 x .8 = 14480 CFS
9285 x .8 = 7428 AC-FT

$$\frac{\text{MAX. SPILLWAY DISCHARGE}}{\text{80\% PMF PEAK INFLOW}} = \frac{12820}{14480} = .885 \quad \text{SAY } 88\%$$

$$\frac{\text{REQD RES STORAGE}}{\text{VOL. OF INFLOW}} = 0.115$$

REQD. RES. STORAGE = .115 x 6964 = 801 AC-FT



APPENDIX C

GEOLOGIC REPORT
AND
STRUCTURAL CALCULATIONS

GEOLOGIC REPORT

Bedrock - Dam and Reservoir

Formation Names: Rose Hill Formation, Mifflintown Formation (including Keefer, Rochester and McKenzie Members).

Lithologies: All three members of the Rose Hill Formation appear to be present. The lower member, on the southeast side of the dam, consists of olive to medium gray weathering, gray clay shale. The total thickness of this unit is more than 600 feet. The Centre Member, also called "iron sandstone" consists of dark reddish gray to dark red sandstone cemented with hematite. It is 70 feet thick. The upper member consists of light brownish gray, shaly claystone. The overlying Keefer Sandstone Member is a light gray to pale yellowish brown sandstone, medium to thick bedded. The sand grains are quartz and are cemented with quartz, or locally, hematite. The Keefer Member is 33 feet thick. The rest of the Mifflintown Formation typically consists of the Rochester Shale Member, 40 feet of gray shale and the McKenzie Member, more than 150 feet of gray shale and gray fine grained limestone. Only gray shale is logged in the core descriptions, so it is probable that only the Rochester Member and lower, shaly part of the McKenzie Member are present.

Structure

Gunter Valley is a tight syncline, whose form is defined by the Tuscarora Quartzite which is exposed on Blue Mountain on the southeast side of the valley, and Kittatinny Mountains on the northwest. The axis of the syncline is apparently on the northwest side of the stream valley, beyond the dam foundation. The foundation borings all indicate northwesterly dips. The axis of the syncline was encountered in digging the Kittatinny Mountain tunnel of the Pennsylvania Turnpike, (Ref.3). It is to be expected that the rocks of the relatively incompetent Rose Hill and Mifflintown Formations are crumpled and faulted in this tight syncline. At least one such fault is probably present, at the contact between the "iron sandstone" and lower shale member of the Rose Hill Formation. Generally the beds under the dam strike N40°E and dip 60° to 70° NW.

Overburden

The overburden here, as indicated by the boring logs is of two types, colluvium and weathered bedrock. The colluvium, material

derived from further up the valley sides, is surprisingly thin, a maximum of 18 feet and generally less than ten feet thick. It consists of sandstone and quartzite boulders in clay and weathered shale matrix.

The depth of weathered bedrock is variable, from four feet to perhaps as much as 25 feet.

Aquifer Characteristics

The Rose Hill and Mifflintown Formations are composed of essentially impermeable rocks. Ground water movement is primarily along bedding planes and fractures. Neither unit yields water to wells in more than small quantities. In some areas the Mifflintown Formation contains limestone and solution openings are possible. No limestone was reported in the foundation borings.

Ground water movement in Gunter Valley is probably primarily along bedding planes and along the faults and fractures that parallel the bedding strike.

Discussion

Because this dam is constructed at right angles to bedding strike, and because there are probably faults and fractures parallel to bedding, it is possible that there is some leakage through bedrock beneath the cutoff trench. The bedrock is reasonably sound, however, and there is little chance that continued leakage would cause any enlargement of the openings.

Sources of Information

1. Conlin, R.R. and Hoskins, D.M. (1962) "The Geology of the Mifflintown Quadrangle" Pa. Geological Survey Atlas A126.

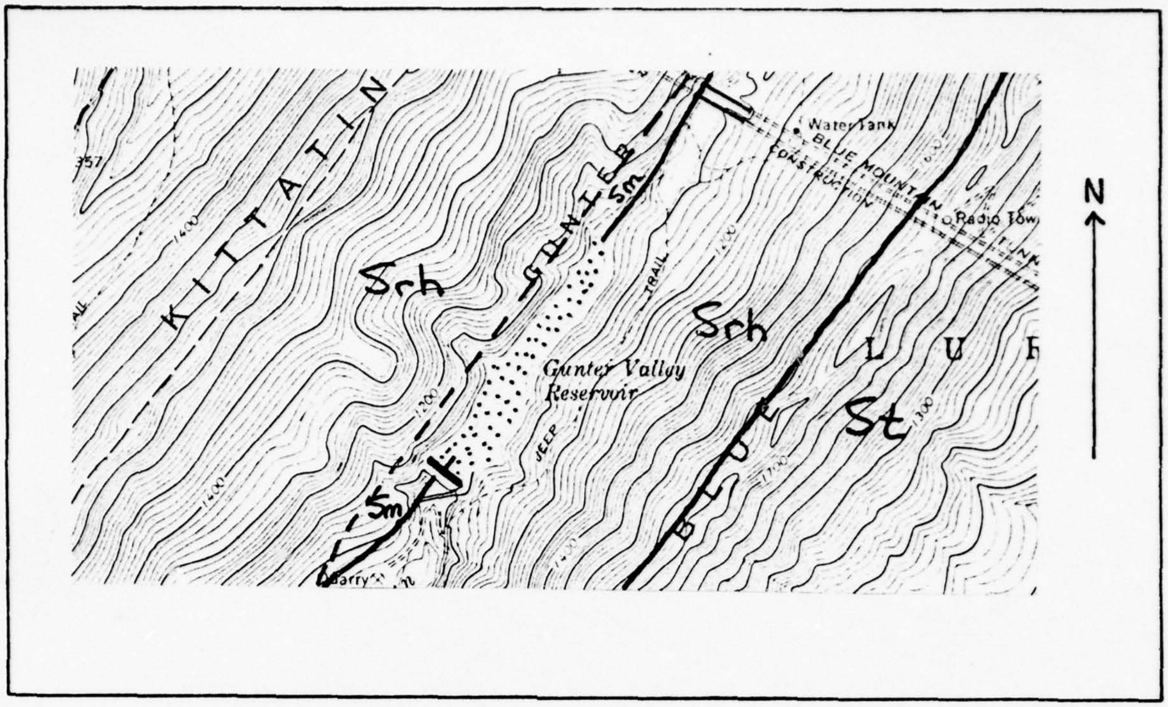
2. Johnston, H.E. (1970) "Ground Water Resources of the Loysville and Mifflintown Quadrangles in South Central Pa." Pa. Geological Survey. Water Resource Report, W-27.

3. Cleaves, A.B. and Stephenson, R.C. "Guidebook to the Geology of the Pennsylvania Turnpike, Carlisle to Irwin". Pa. Geological Survey Bulletin G-24.

4. Geologic Map of the Doylesburg Quadrangle, open file. Pa. Geological Survey, Harrisburg, Pa.

5. Logs of Borings in File.

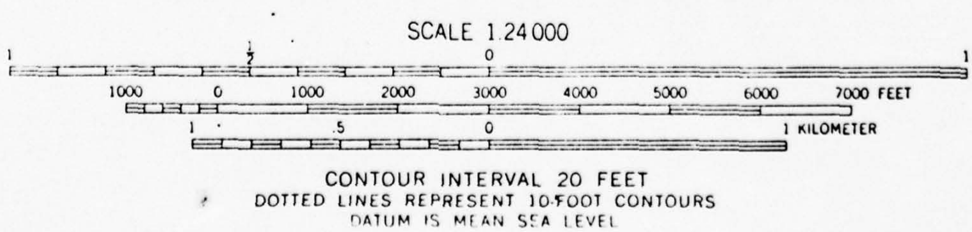
6. Air Photographs, scale 1:24,000 dated 1964.

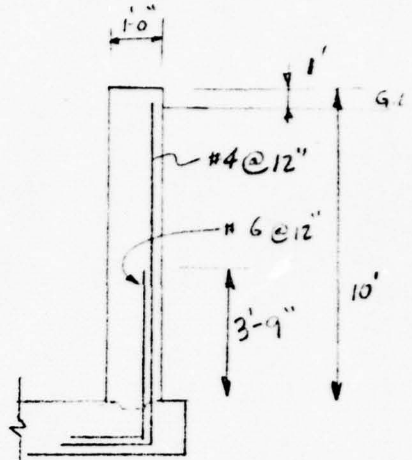


(geology from open file maps, Pa. Geol. Surv.)

KEY

Sm	Mifflintown Fm.
Srh	Rose Hill Fm.
St	Tuscarora Fm.
- - - -	fault





RETAINING WALL

The wall retains an earth fill of 9.0' along with free Ground water. Under worst conditions the water stands at the same height as the fill.

Saturated weight of soil is taken as 140 pcf − 62.4 pcf = 77.6 pcf. with a ∠ of repose of 35°

$\therefore P_a(\text{Soil}) = 77.6\left[\dfrac{1-\sin 35}{1+\sin 35}\right] = 77.6\left(\dfrac{0.426}{1.574}\right) = 21\text{ psf}.$

∴ O.T. Moment.

1) Soil $= \frac{1}{2}(0.021)(9.0)^2 = 0.851 \times 3 = 2.552'^k$

2) Water $= \frac{1}{2}(0.0624)(9.0)^2 = \underline{2.527} \times 3 = \underline{7.582}$

3.378^k $\qquad 10.134'^k$

Vertical load:-

$1 \times 10' \times .15 = 1.5^k \times \frac{1}{2}(1) = 0.75'^k$

$\therefore R = \frac{1}{1.5}(10.134 - 0.75) = 6.256' = 75.07''$

$d = 12 - 1.5 - 0.5 - 0.37 = 9.63''$

$e = 75.07 + 9.63 = 84.7'' = 7.058'$

$\therefore \dfrac{e}{d} = \dfrac{84.7}{9.63} = 8.8$ $\qquad$ Use $i = 1.112$ & $f_s = 20$ ksi

$\therefore A_s = \dfrac{1.5 \times 7.058}{1.112 \times 9.63 \times 1.44} = 0.68\text{ in}^2$

Provided #6 @ 12" } $A_s = 0.64\text{ in}^2/'$ $\approx 0.68\text{ in}^2$
#4 @ 12" }

(6.5% overstress.)

Check section @ 2' above ftg.

Vertical:

$1' \times 8' \times 0.15 = 1.0' \times (\frac{1}{2})(1) = 0.5'^k$

$d = 12 - 1.5 - 0.5 - 0.25$

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O.T Moment

1) Soil $\frac{1}{2}(.021)(7)^2 = 0.515^k \times 7/3 = 1.201'^k$

2) Water $\frac{1}{2}(.0624)(7)^2 = \underline{1.529^k} \times 7/3 = \underline{3.567'^k}$

$2.044^k \qquad 4.768'^k$

$\therefore R = \frac{1}{1.0}\left(4.768 - 0.5\right) = 4.268' = 51.216$

$\therefore e = 51.216'' + 9.75 = 60.966'' = 5.08'$

$\therefore \frac{e}{d} = \frac{60.966}{9.75} = 6.25$

$i = 1.164$

$\therefore A_s = \frac{1.0 \times 5.08}{1.44 \times 1.164 \times 9.75} = 0.31\ in^2/'$

At 3'-9" above Ftg.

O.T. Mom

1) Soil $\frac{1}{2}(.021)(5.25)^2 = 0.289 \times 5.25/3 = 0.506'^k$

2) Water $\frac{1}{2}(.0624)(5.25)^2 = \underline{0.860} \times 5.25/3 = \underline{1.505'^k}$

$1.149^k \qquad 2.011'^k$

Vertical Load

$1 \times .15 \times 6.25 = 0.94^k \times 0.5 = 0.47'^k$

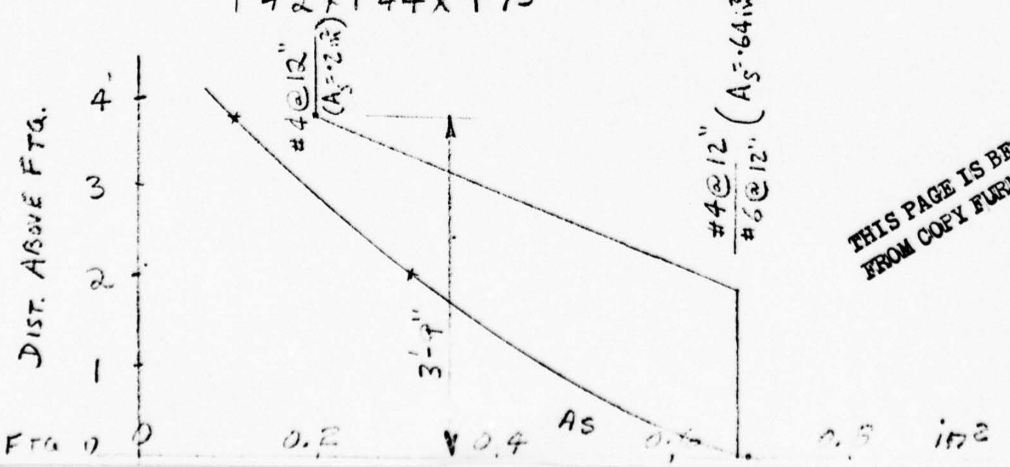
$d = 9.75''$ $\qquad R = \frac{1}{0.94}(2.011 - 0.47) = 1.64'$

$\therefore E = 1.64 \times 12 + 9.75 = 29.43'' = 2.453'$

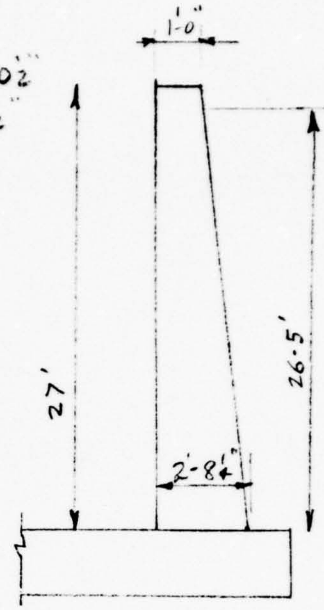
$\therefore \frac{e}{d} = \frac{29.43}{9.75} = 3.02$

$i = 1.42'$

$\therefore A_s = \frac{0.94 \times 2.453}{1.42 \times 1.44 \times 9.75} = 0.116\ in^2$



CHECK DEFLECTION OF CANTILEVER WALL:



The wall height varies from 28'-6" to 23'-10½"
Design for a ht. $=(23'\text{-}10\tfrac{1}{2}")+\frac{2}{3}(4'\text{-}7\tfrac{1}{2}")=26'\text{-}11\tfrac{1}{2}"$
$\sim 27'$

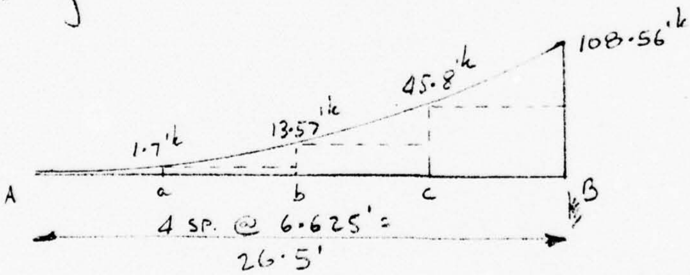
O.T. Mom:-
Soil active pressure = 35 psf.
$\therefore \frac{1}{2}(.035)(26.5)^2 = 12.289^k \times 26.5/3 = 108.56'^k/'$

Concrete used = 4,000 psi.
$f_s = 20,000$ psi.
$E_c = w^{1.5} \times 33\sqrt{f'_c} = 3.834 \times 10^6$ psi.
$= 5.5213 \times 10^5$ ksf.

Consider the concrete section as gross section @ top & bottom:
For the prel. neglect reinforcement.

At top: $I_{xx} = \frac{1}{12}(12)(12)^3 = 1728 \text{ in}^3/' = 0.0833 \text{ ft}^4$
At bottom: $I_{xx} = \frac{1}{12}(12)(32.25)^3 = 33,542 \text{ in}^4 = 1.6176 \text{ ft}^4/'$

$\therefore$ M/EI Diagram



Section @ 'a' : $d = 1.4453'$ $\therefore I_{xx} = \frac{1}{12}(1)(1.4453)^3 = 0.2516 \text{ ft}^4$

Section @ 'b' : $d = 1.8594'$ $I_x = \frac{1}{12}(1)(1.8594)^3 = 0.5357 \text{ ft}^4$

Section @ c : $d = 2.2734'$ $I_x = \frac{1}{12}(1)(2.2734)^3 = 0.9792 \text{ ft}^4$

Use average I_{xx} between sections.

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$$E\times\Delta = \left[\frac{\tfrac{1}{3}(1.7\times6.625)(\tfrac{3}{4}\times6.625)}{\tfrac{1}{2}(0.0833+0.2516)}\right]$$

$$+\left[\frac{\left[\tfrac{1}{3}(11.87\times6.625)(\tfrac{3}{4}\times6.625+6.625)\right]+\left[1.7\times6.625\times9.9375\right]}{\tfrac{1}{2}(0.2516+0.5357)}\right]$$

$$+\left[\frac{\left\{\tfrac{1}{3}(32.23)(6.625)(\tfrac{3}{4}\times6.625+13.25)\right\}+\left\{13.37\times6.625\times16.5625\right\}}{\tfrac{1}{2}(0.5357+0.9792)}\right]$$

$$+\left[\frac{\left\{\tfrac{1}{3}(62.76)(6.625)(\tfrac{3}{4}(6.625)+19.875)\right\}+\left\{45.8\times6.625\times23.1875\right\}}{\tfrac{1}{2}(0.9792+1.6176)}\right]$$

$$\Delta E = \frac{18.6535}{0.1675}+\frac{303.906+111.921}{0.3937}+\frac{1296.712+1467.044}{0.7575}+\frac{3443.22+7035.667}{1.2984}$$

$$= 111.398+1056.337+3648.764+8070.615 = 12887.114$$

$$\therefore\ \Delta = \frac{12887.114}{552130} = 0.02334' = 0.28'' \sim \frac{5}{16}''$$

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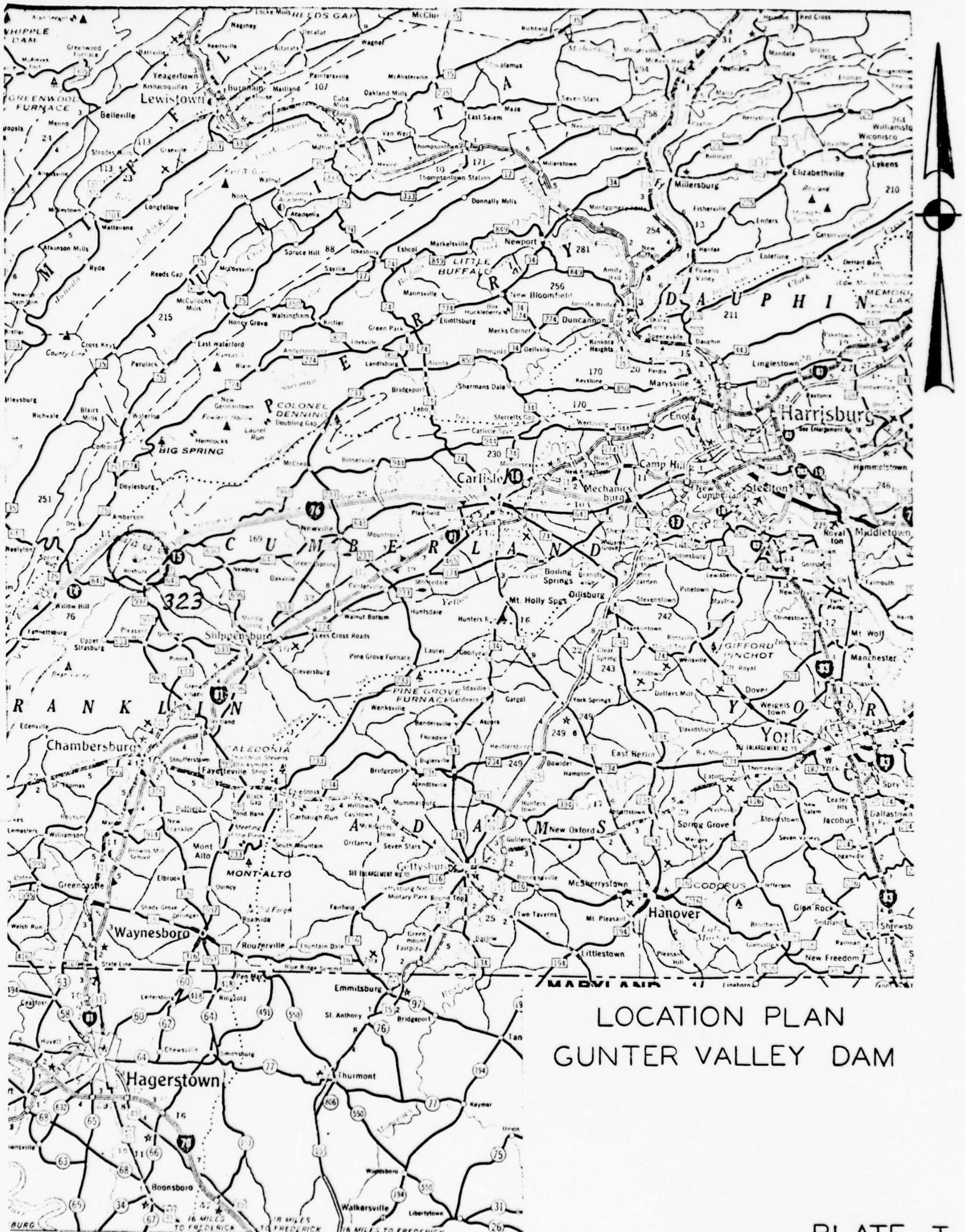
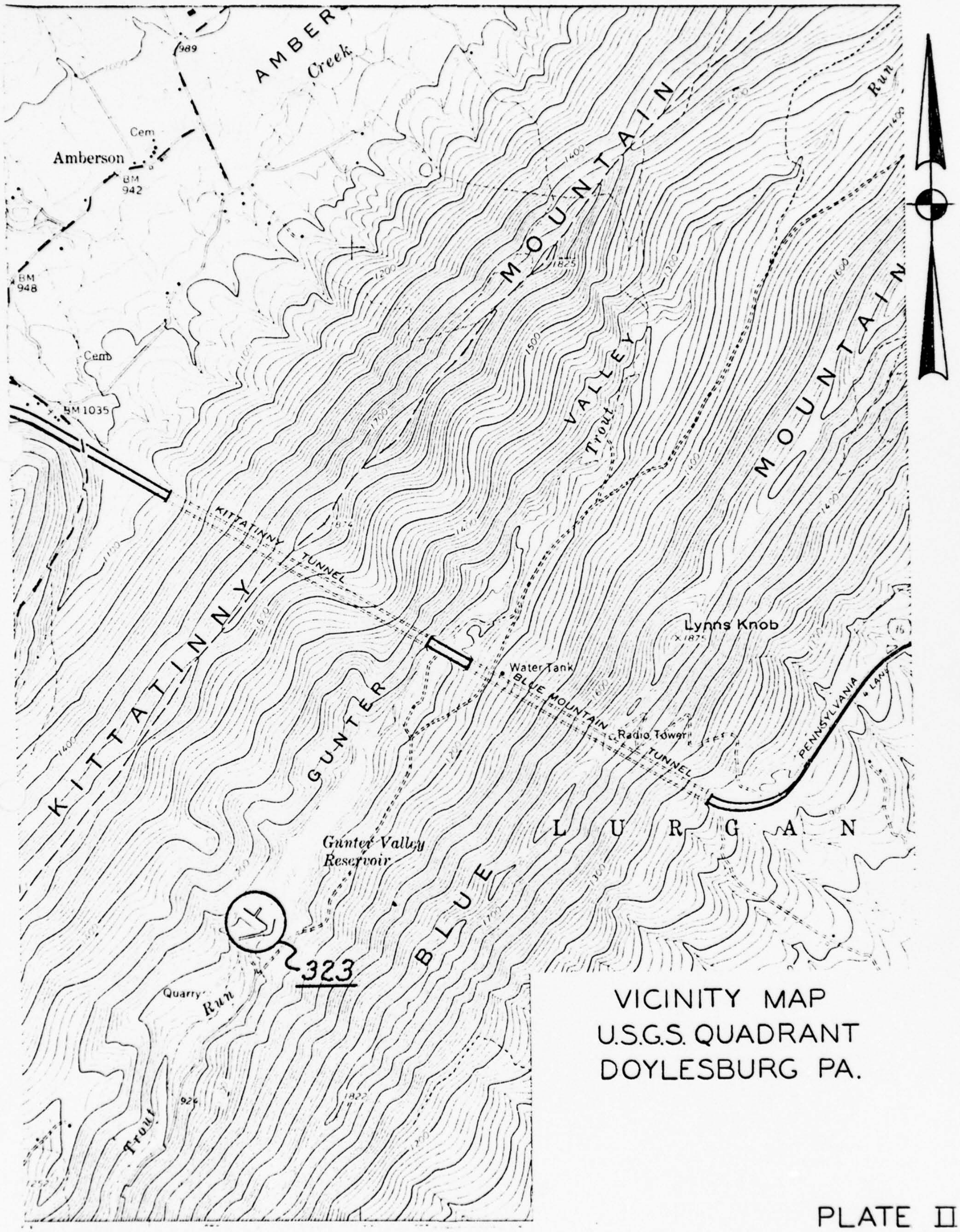
APPENDIX D

LOCATION, PHOTOGRAPHS & DESIGN DRAWINGS



LOCATION PLAN
GUNTER VALLEY DAM

PLATE I

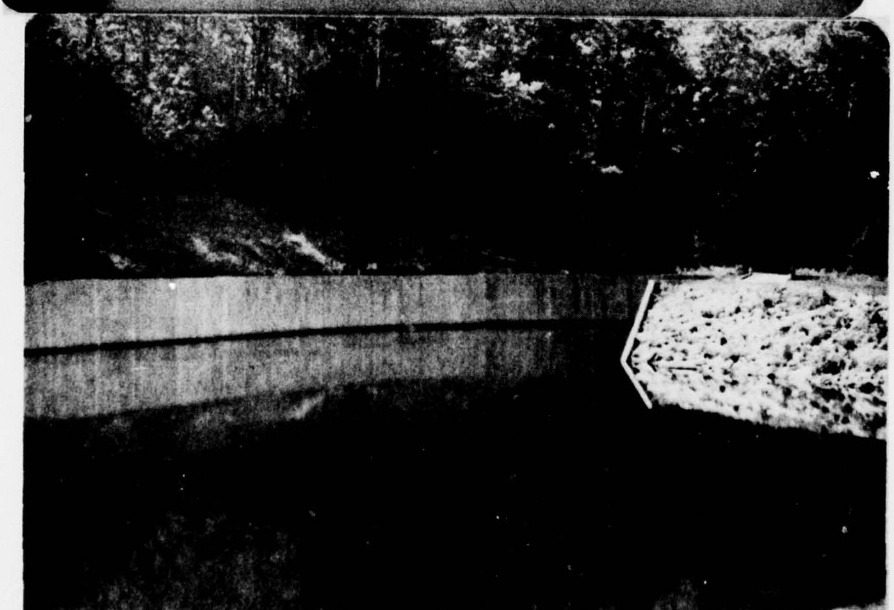


VICINITY MAP
U.S.G.S. QUADRANT
DOYLESBURG PA.

PLATE II

Right Abutment

Reservoir



Forebay

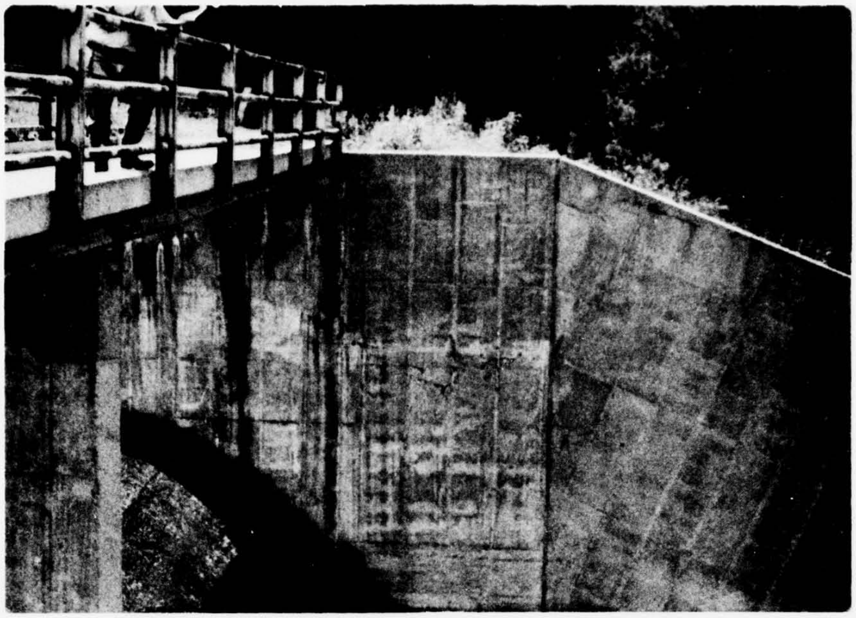
PLATE III

Spillway &
Bridge

Spillway Channel

Spillway

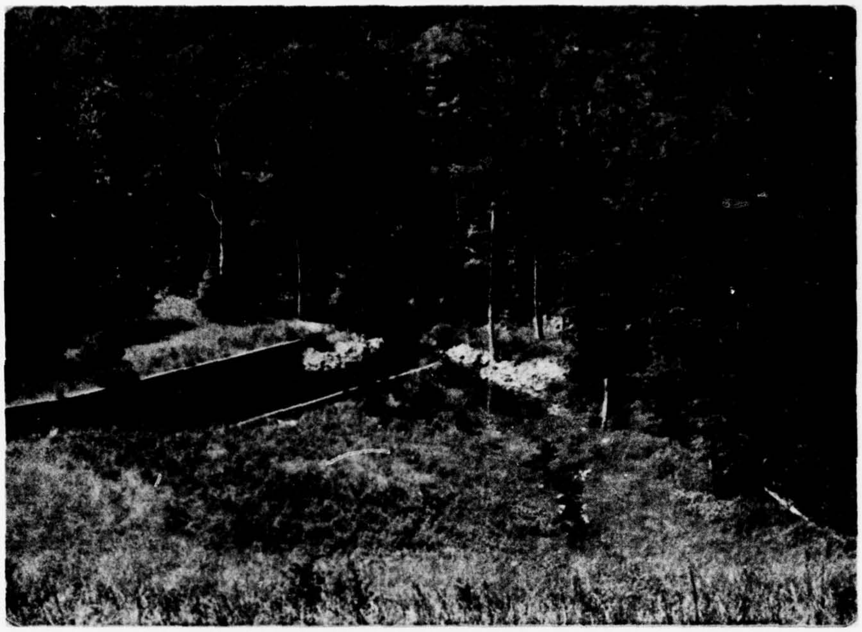
PLATE IV



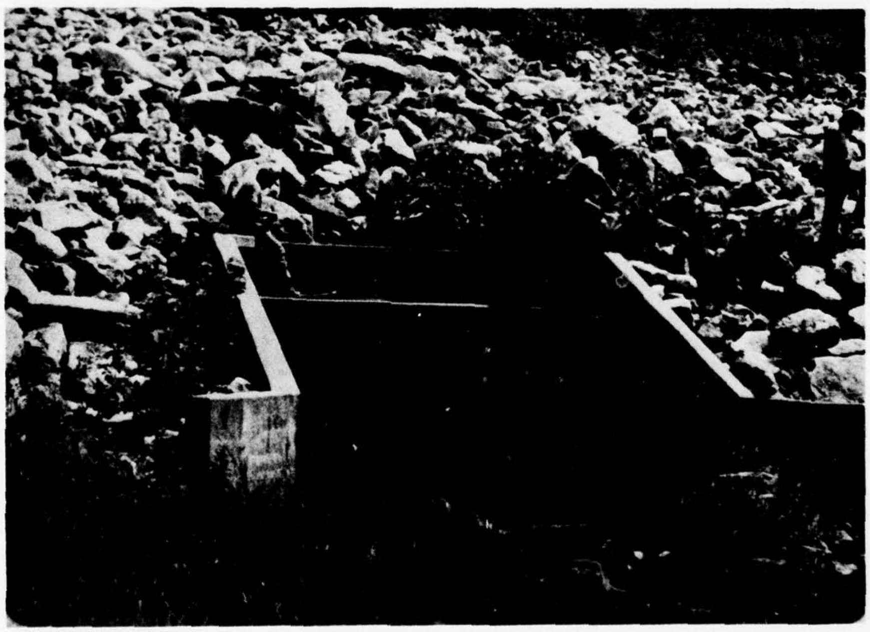
Wall Cracking

Wall Movement Spillway Wall

PLATE V

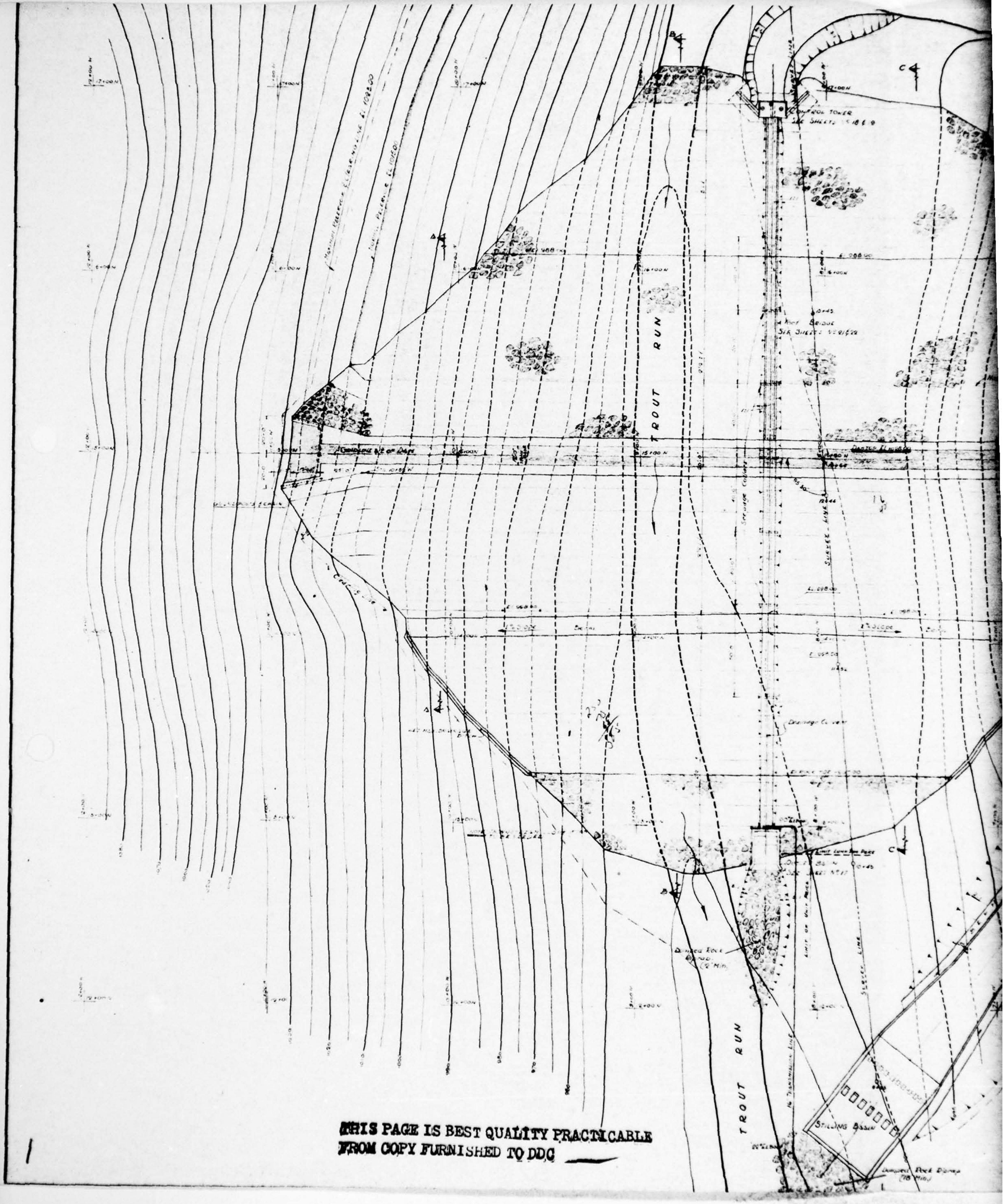


Downstream Channel



Outlet Structure

PLATE VI


TROUT RUN
TROUT RUN
TROUT RUN
CONTROL TOWER
Foot Bridge
Stilling Basin
Dumped Rock Riprap
Drainage Culvert
SPILLWAY CHANNEL

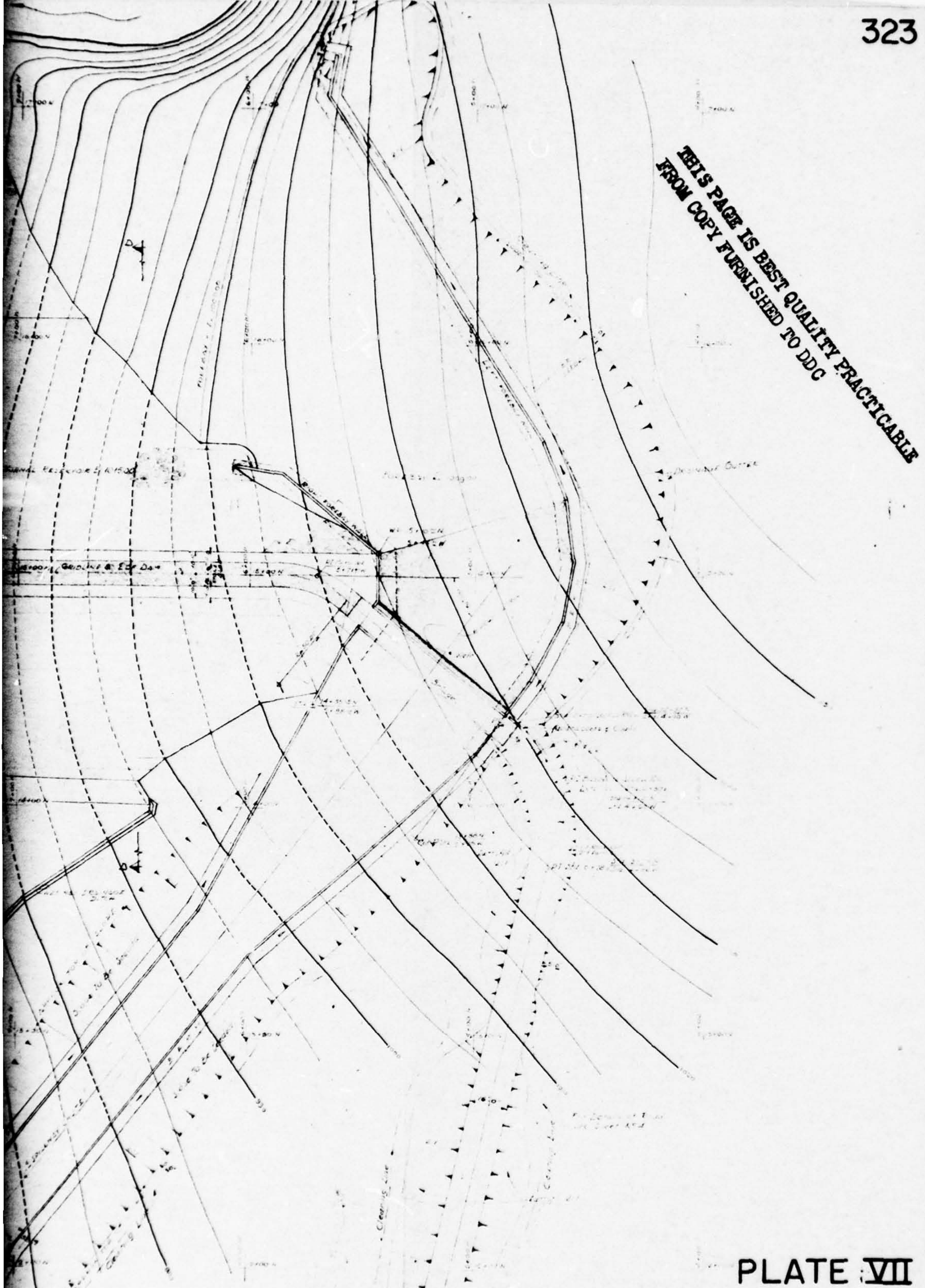


PLATE VII

SEAL	SHIPPENSBURG BOROUGH AUTHORITY SHIPPENSBURG, PENNA.
	GENERAL PLAN EARTHFILL DAM
	GLACE & GLACE Inc. CONSULTING ENGINEERS HARRISBURG, PENNSYLVANIA
SEAL / MARK / REVISIONS / DATE	DRAWN BY / TRACED BY / CHKD BY — APPROVED: ENGINEER 2 DATE — SCALE 1" = 25' — FILE CODE 5806 — SHEET NO. 7 OF 24

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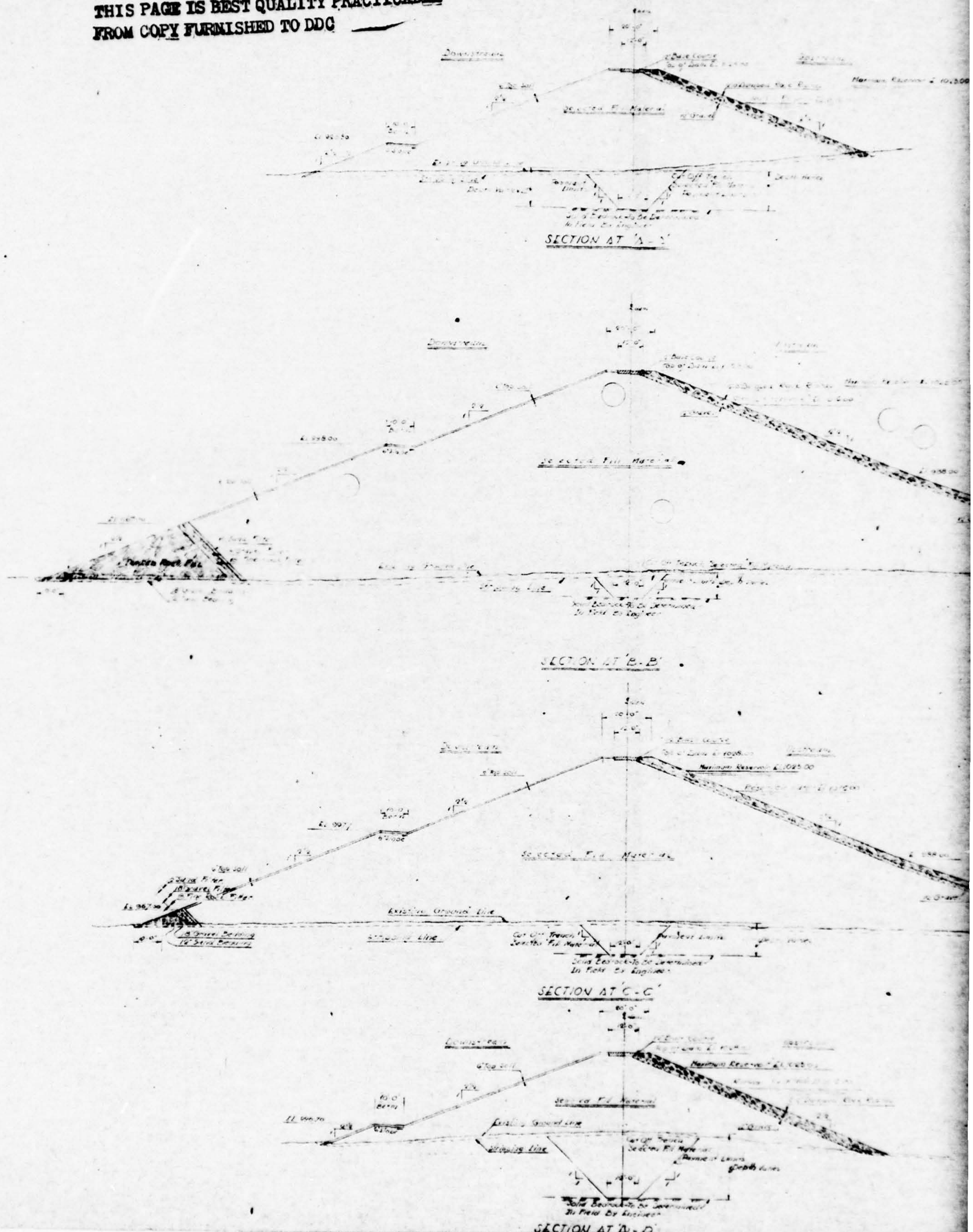


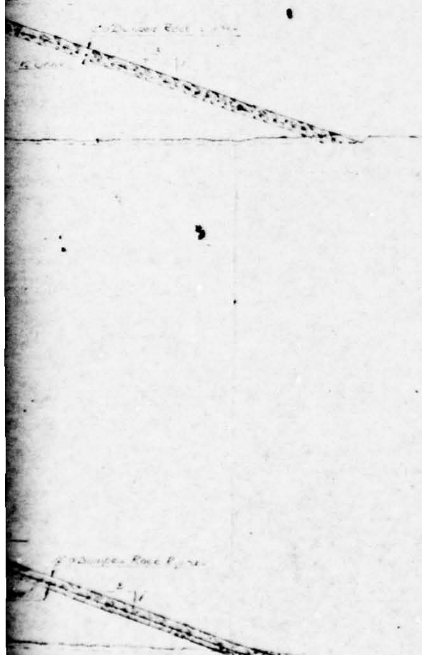


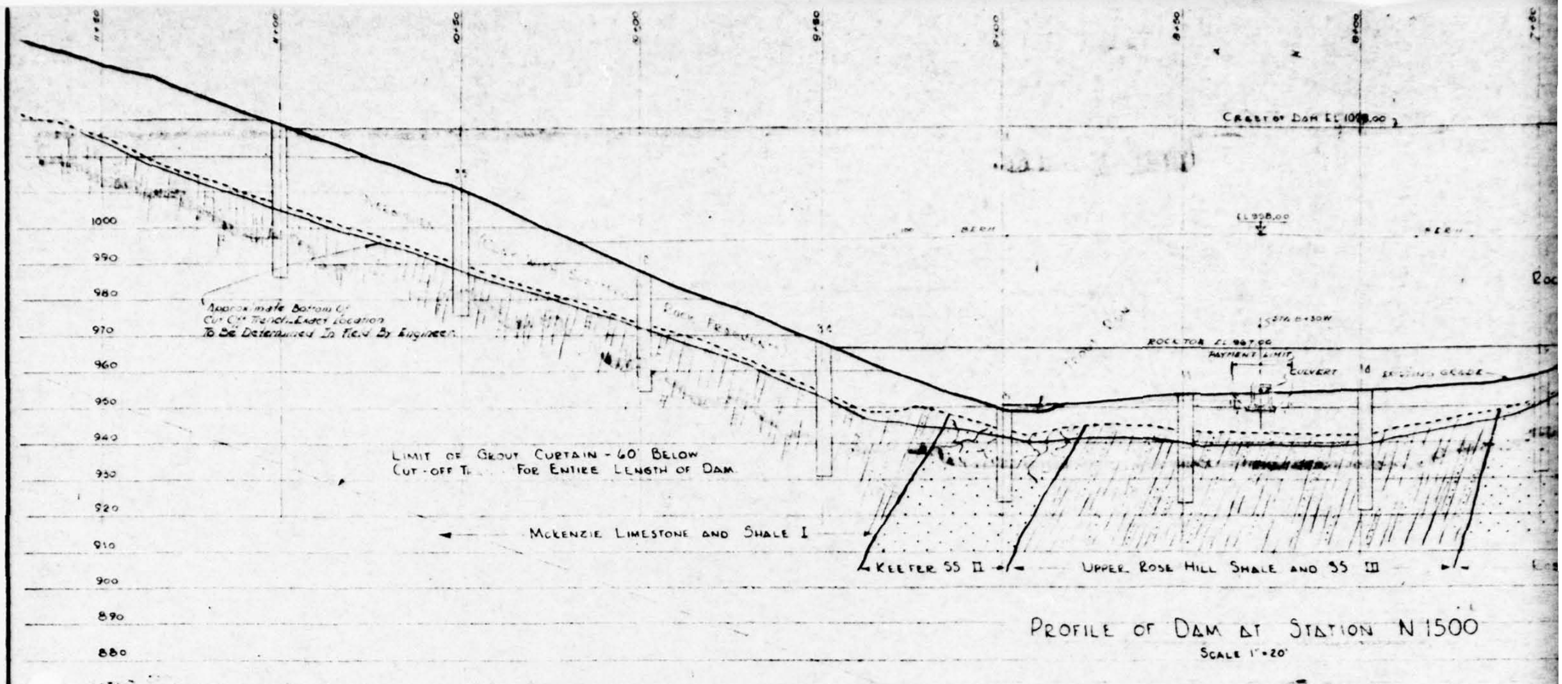
PLATE VIII

SHIPPENSBURG BOROUGH AUTHORITY
SHIPPENSBURG, PENNA.

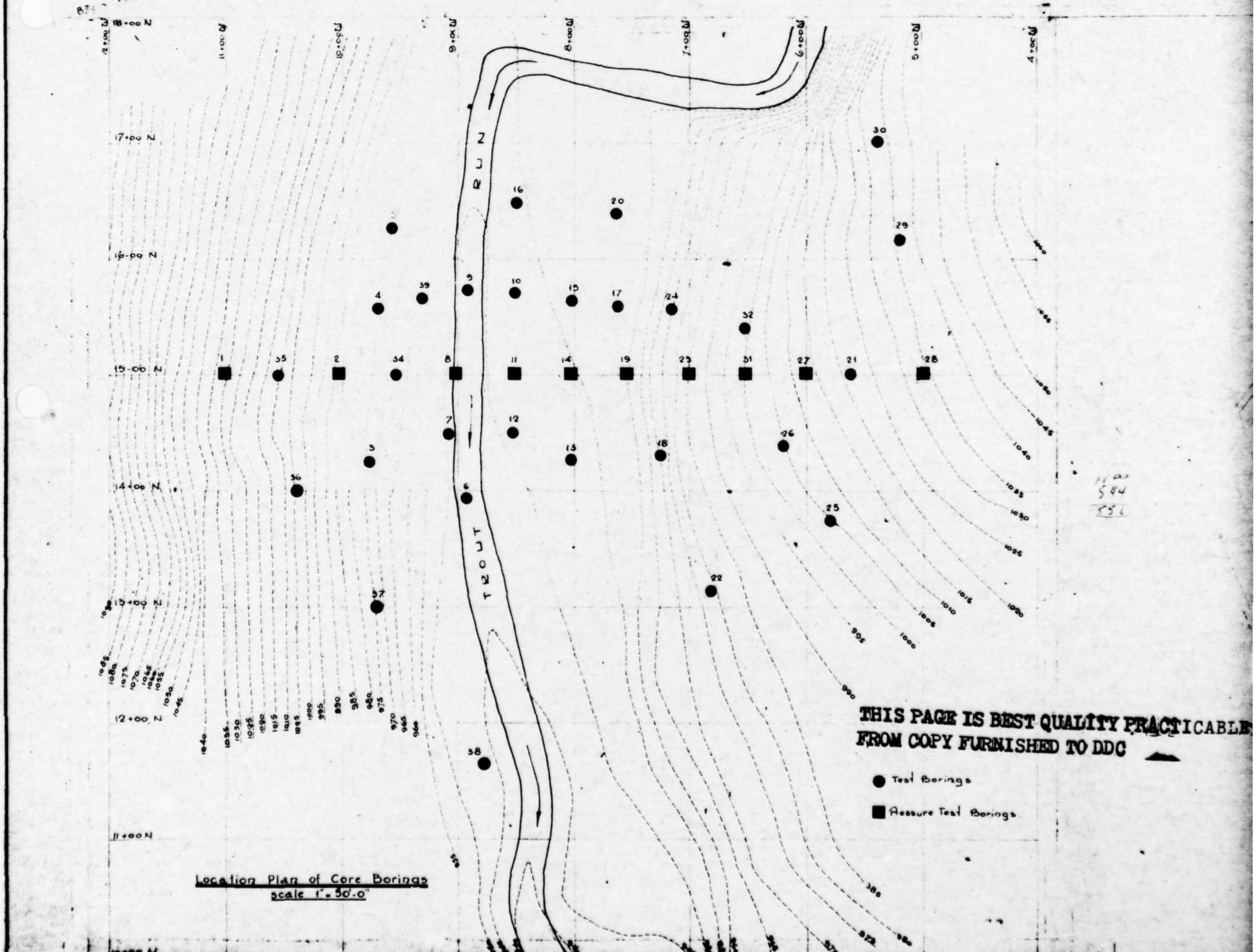
TYPICAL DAM SECTIONS

GLACE & GLACE Inc.
CONSULTING ENGINEERS
HARRISBURG, PENNSYLVANIA

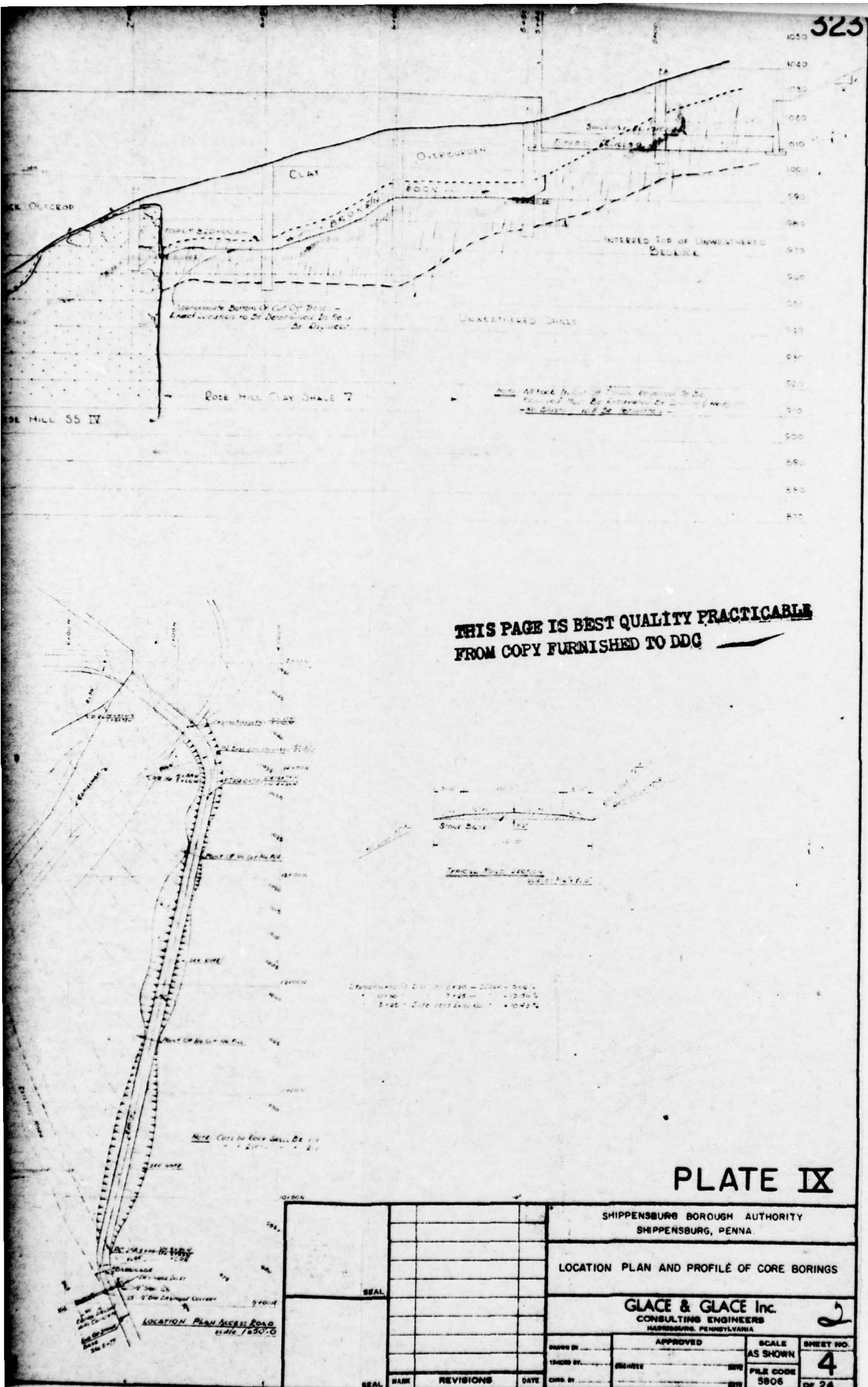
SEAL	MARK	REVISIONS	DATE	APPROVED	SCALE 1" = 20'
SEAL					SHEET NO. 8 OF 24



PROFILE OF DAM AT STATION N 1500
Scale 1" = 20'



Location Plan of Core Borings
scale 1" = 50'-0"


Clay
Overburden
Rock Outcrop
Inferred Top of Unweathered Bedrock
Unweathered Shale
Rose Hill Clay Shale 7
Rose Hill SS IV
Typical Road Section
Location Plan Access Road
PLATE IX
SHIPPENSBURG BOROUGH AUTHORITY
SHIPPENSBURG, PENNA.
LOCATION PLAN AND PROFILE OF CORE BORINGS
GLACE & GLACE Inc.
CONSULTING ENGINEERS
HARRISBURG, PENNSYLVANIA
SEAL
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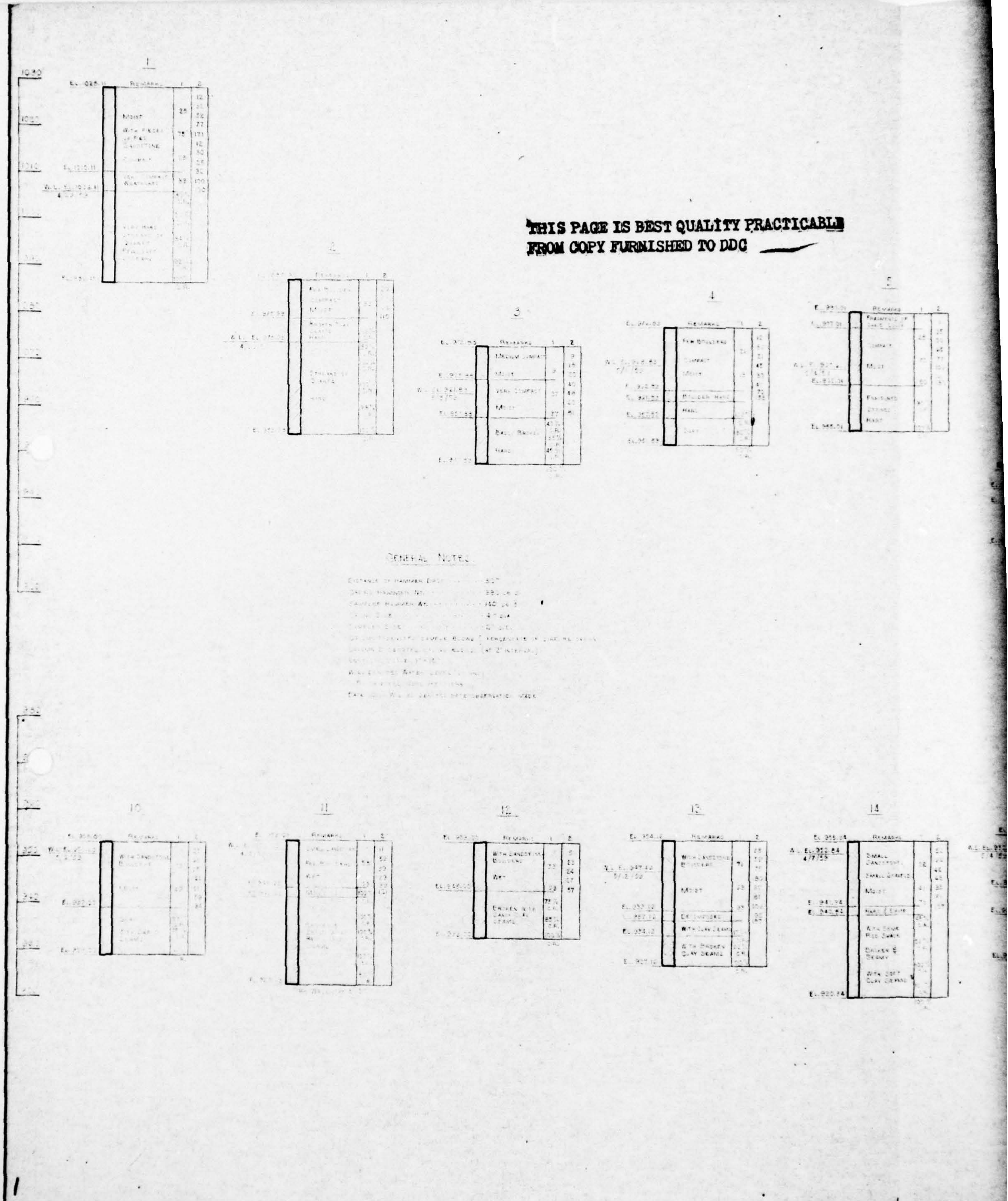
REVISIONS
DATE
APPROVED
SCALE
AS SHOWN
SHEET NO.
4
OF 24
FILE CODE
5806


1
2
3
4
5
10
11
12
13
14
Remarks
Moist
Compact
Few Boulders
Hard
General Notes

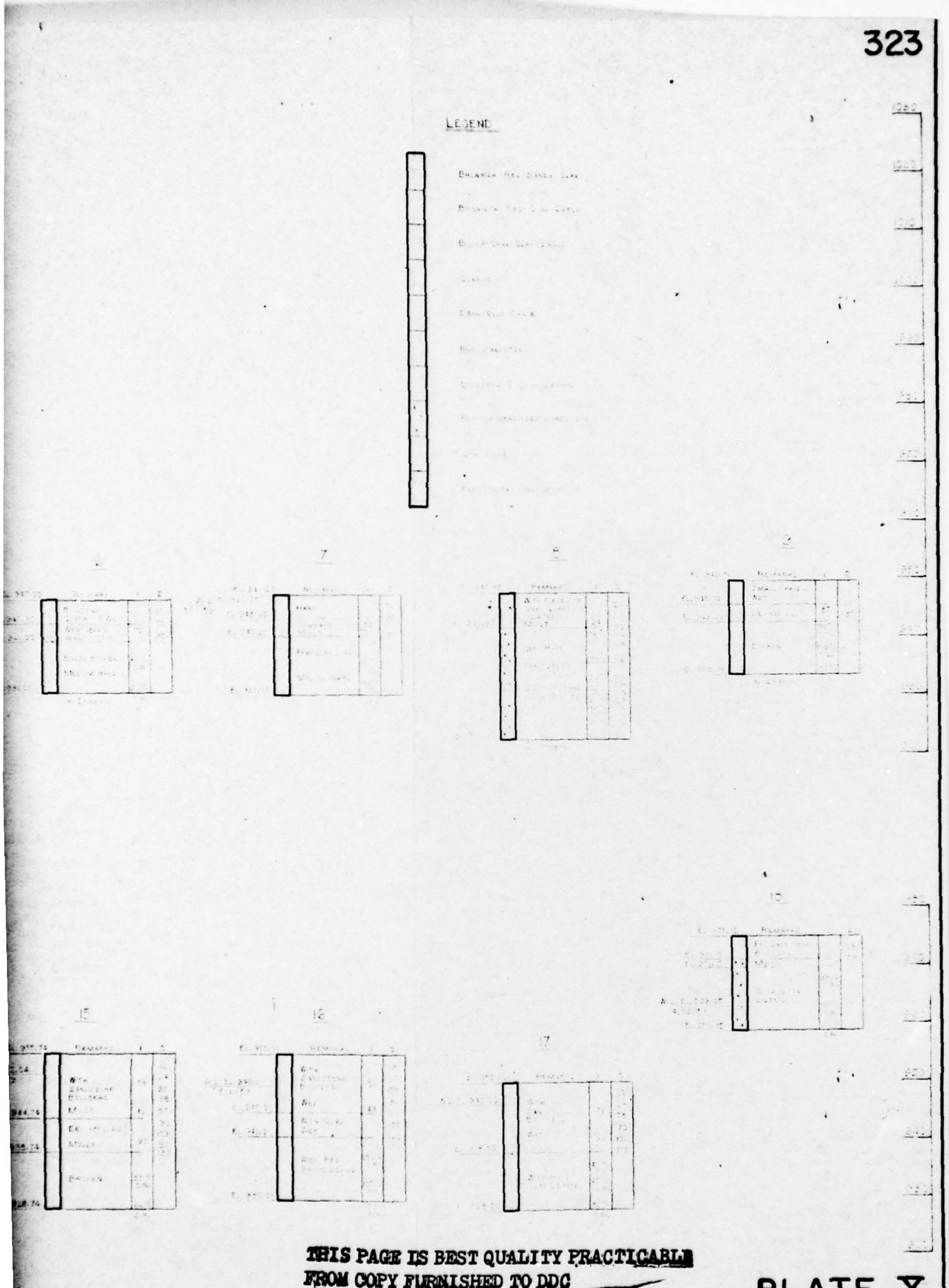


PLATE X

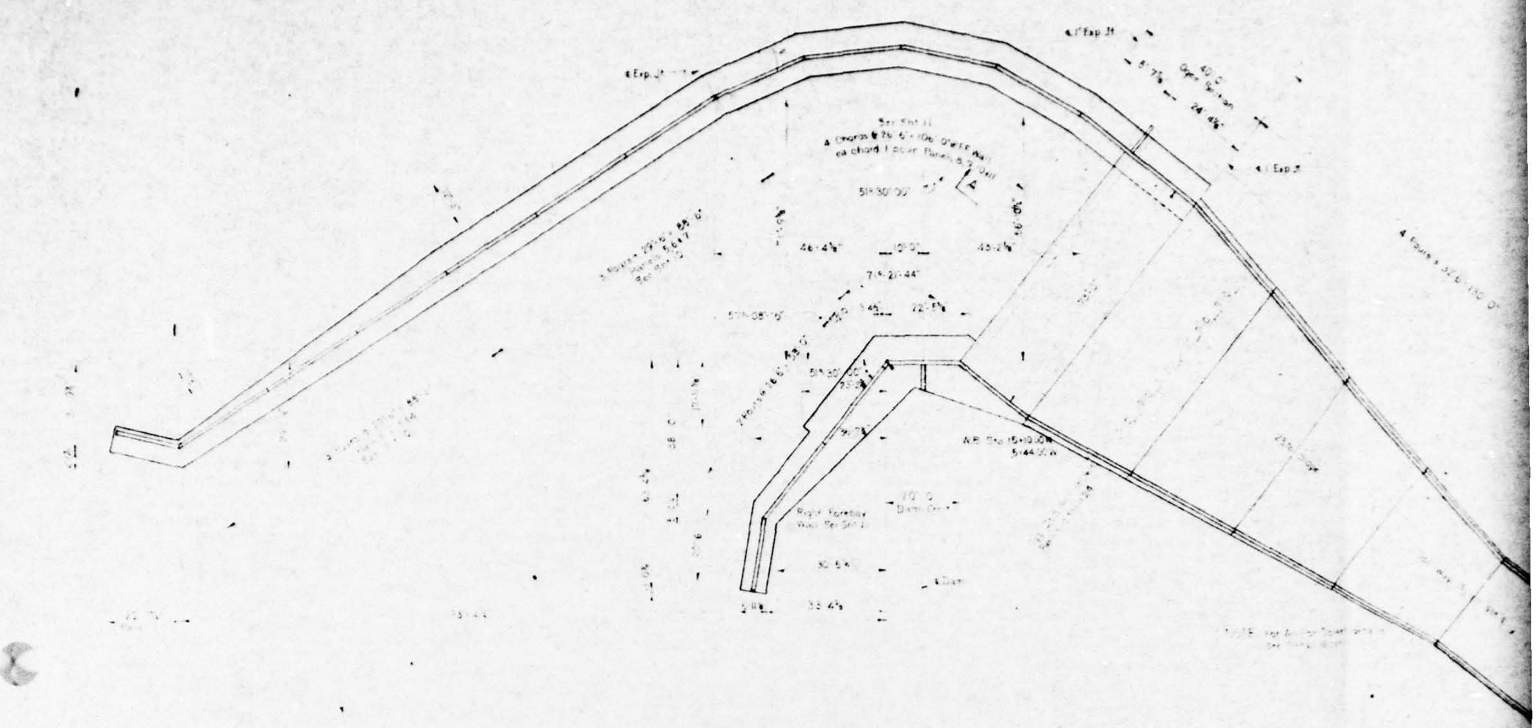
SHIPPENSBURG BOROUGH AUTHORITY
SHIPPENSBURG, PENNA.

CORE BORINGS NO. 1 TO 18

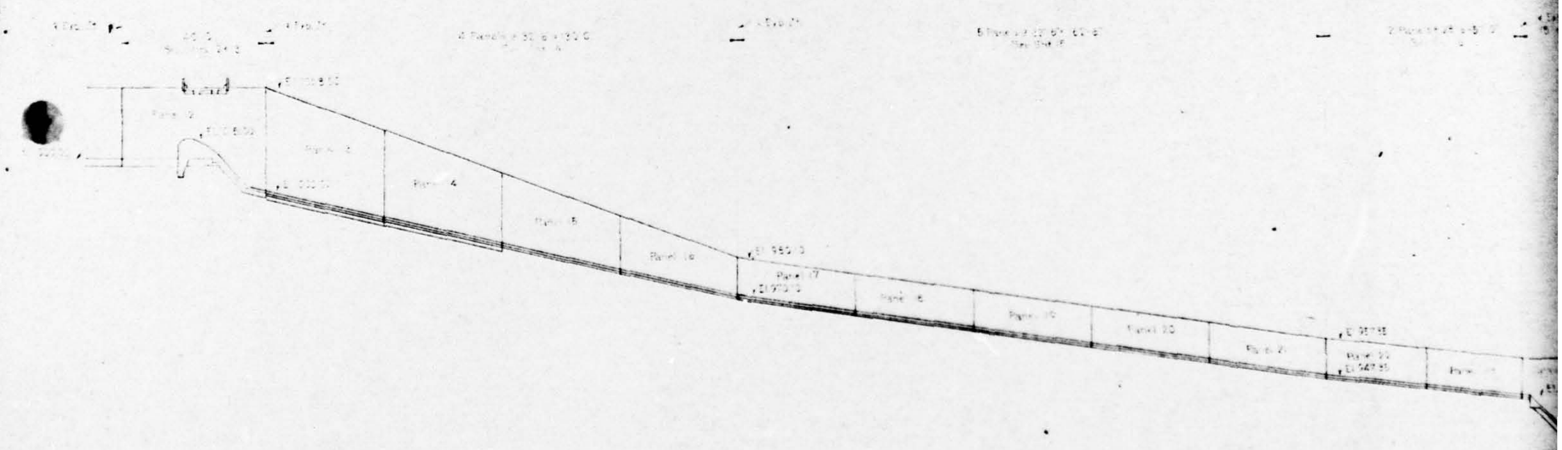
GLACE & GLACE Inc.
CONSULTING ENGINEERS
HARRISBURG, PENNSYLVANIA

2

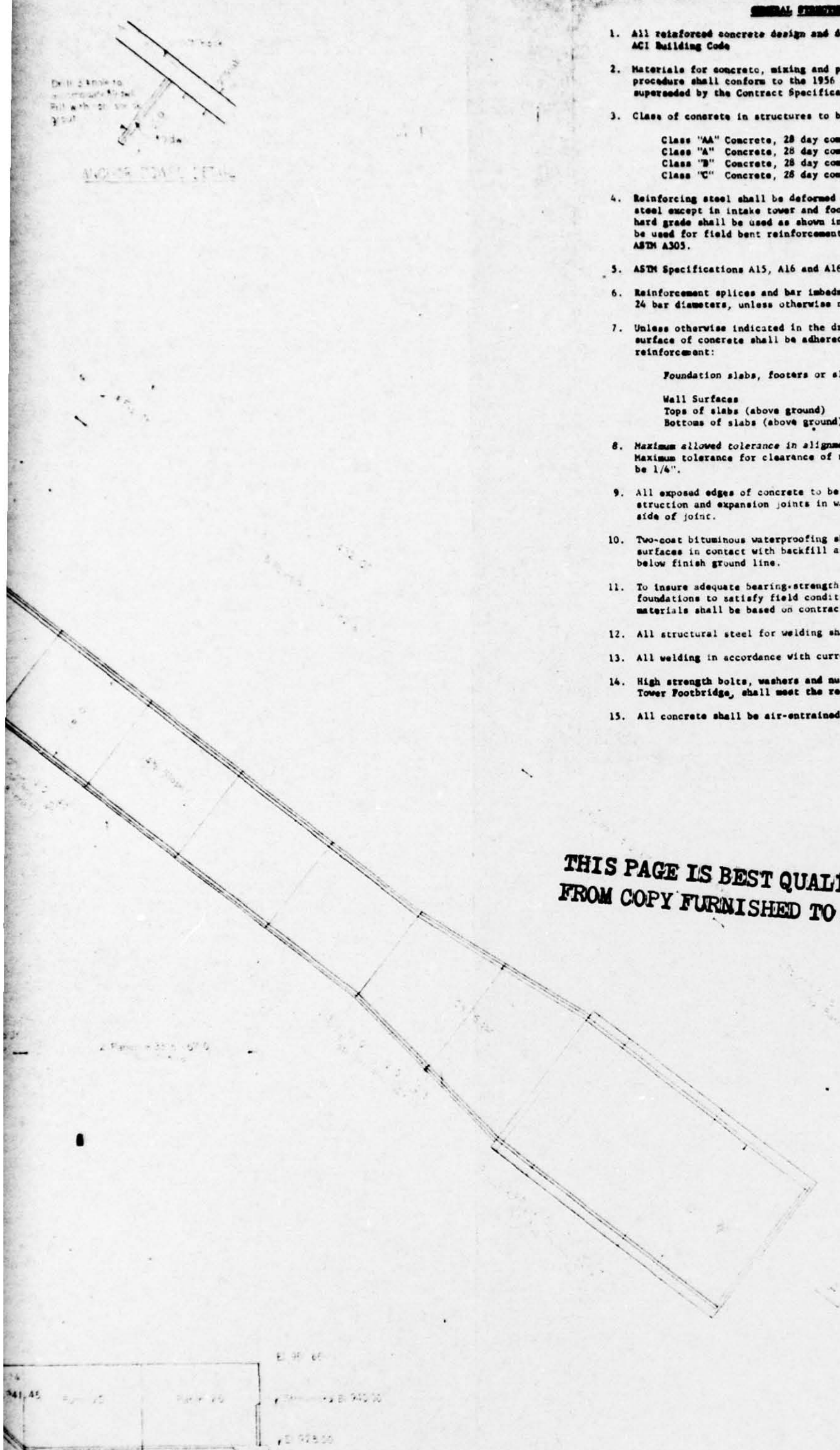
SEAL	MARK	REVISIONS	DATE	DRAWN BY	TRACED BY	CHKD. BY	APPROVED	SCALE	FILE CODE	SHEET NO.
								1" = 10'	5806	5 OF



PLAN



LONGITUDINAL SECTION A-A



GENERAL STRUCTURAL NOTES

1. All reinforced concrete design and details in accordance with 1956 ACI Building Code

2. Materials for concrete, mixing and placing of concrete and construction procedure shall conform to the 1956 ACI Building Code, except where superseded by the Contract Specifications.

3. Class of concrete in structures to be as shown in drawings.

   Class "AA" Concrete, 28 day compressive strength f'c = 5,000 p.s.i.
   Class "A" Concrete, 28 day compressive strength f'c = 4,000 p.s.i.
   Class "B" Concrete, 28 day compressive strength f'c = 2,500 p.s.i.
   Class "C" Concrete, 28 day compressive strength f'c = 2,000 p.s.i.

4. Reinforcing steel shall be deformed and of intermediate or hard grade steel except in intake tower and footbridge deck, pier and abutment where hard grade shall be used as shown in drawings. Hard grade steel shall not be used for field bent reinforcement. Bar deformations shall conform to ASTM A305.

5. ASTM Specifications A15, A16 and A160 shall govern.

6. Reinforcement splices and bar imbedment lengths shall be a minimum of 24 bar diameters, unless otherwise noted in drawings.

7. Unless otherwise indicated in the drawings the following clearances to surface of concrete shall be adhered to in detailing and placing reinforcement:

Foundation slabs, footers or slabs on fill	-	3" bottom
	-	2" Top and sides
Wall Surfaces	-	1-1/2"
Tops of slabs (above ground)	-	1"
Bottoms of slabs (above ground)	-	3/4"

8. Maximum allowed tolerance in alignment of wall surfaces shall be 1/2". Maximum tolerance for clearance of reinforcement to face of concrete shall be 1/4".

9. All exposed edges of concrete to be chamfered 3/4" x 3/4". Exposed construction and expansion joints in walls shall be chamfered 3/4" x 3/4" each side of joint.

10. Two-coat bituminous waterproofing shall be applied to all formed concrete surfaces in contact with backfill as shown in drawings and shall stop 1'-0" below finish ground line.

11. To insure adequate bearing strength, the Engineer may order revisions in foundations to satisfy field conditions. Payment for additional labor and materials shall be based on contract unit prices.

12. All structural steel for welding shall meet requirements of ASTM A373-58T.

13. All welding in accordance with current AWS Specifications.

14. High strength bolts, washers and nuts, to be used in field assembly of Intake Tower Footbridge, shall meet the requirements of ASTM A325.

15. All concrete shall be air-entrained, except where otherwise noted in drawings.

PLATE XI

SHIPPENSBURG BOROUGH AUTHORITY
SHIPPENSBURG, PENNA.

GENERAL PLAN OF SPILLWAY

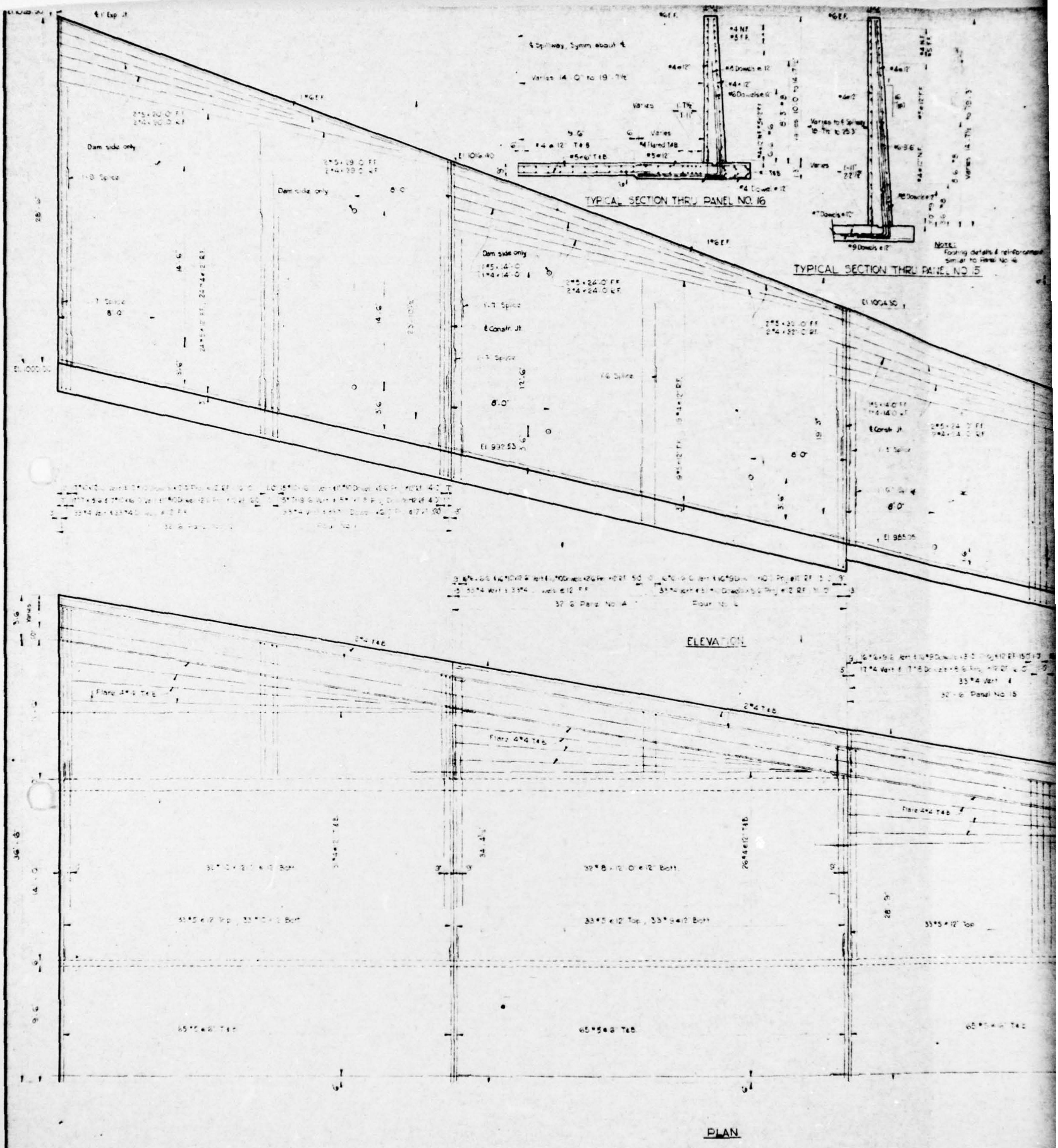
GLACE & GLACE Inc.
CONSULTING ENGINEERS
HARRISBURG, PENNSYLVANIA

SEAL

MARK	REVISIONS	DATE

DRAWN BY
TRACED BY
CHKD BY
APPROVED
ENGINEER DATE

SCALE	SHEET NO.
1"=20'	9
FILE CODE 6003	


TYPICAL SECTION THRU PANEL NO. 16
TYPICAL SECTION THRU PANEL NO. 15
Note: Footing details & reinforcement similar to Panel No. 16
Dam side only
ELEVATION
PLAN

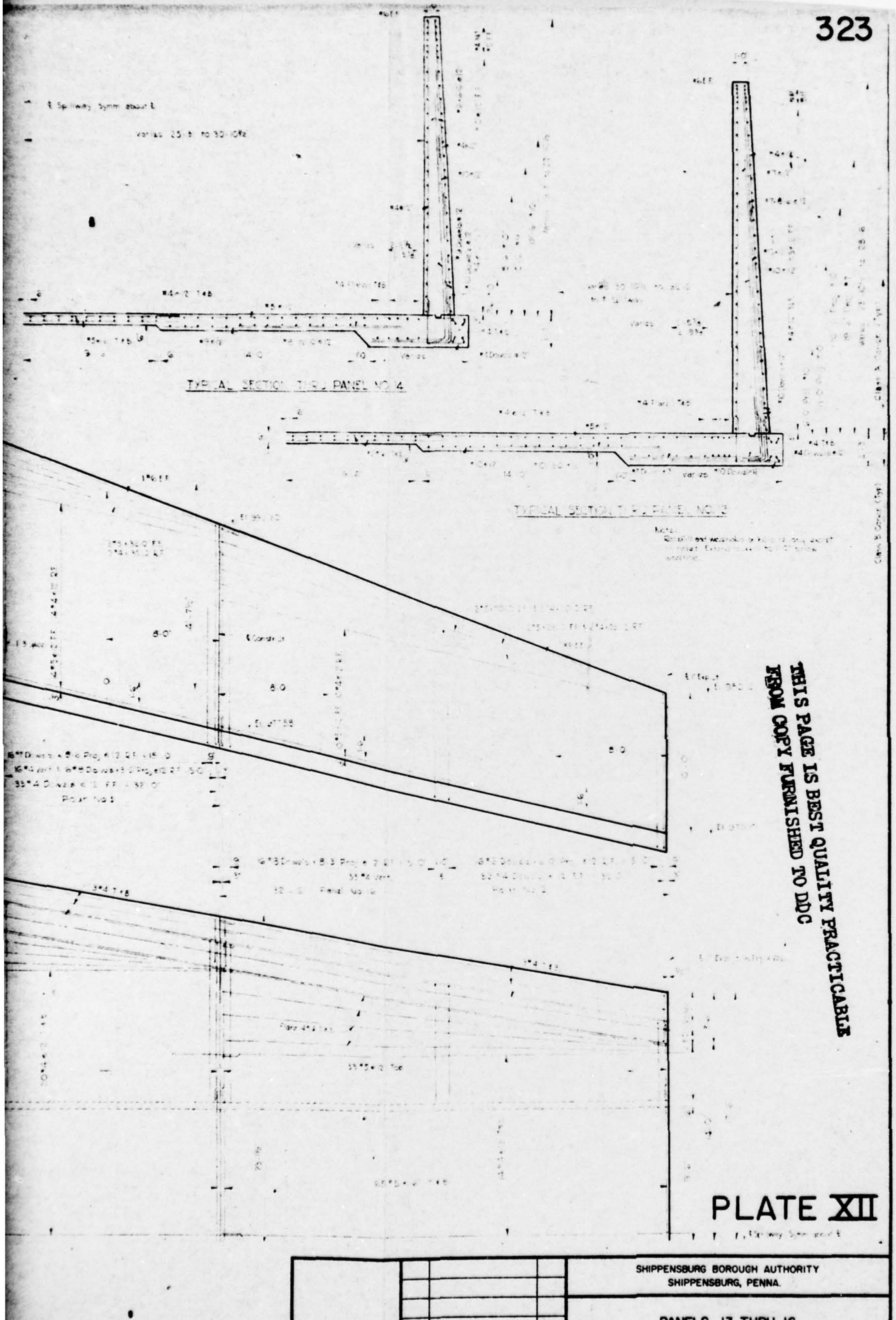


PLATE XII

SHIPPENSBURG BOROUGH AUTHORITY
SHIPPENSBURG, PENNA.

PANELS 13 THRU 16

GLACE & GLACE Inc.
CONSULTING ENGINEERS
HARRISBURG, PENNSYLVANIA

SEAL	MARK	REVISIONS	DATE	DRAWN BY	TRACED BY	CHKD. BY	APPROVED	SCALE	SHEET NO.
SEAL							ENGINEER / DATE	1/4" = 1'-0" / FILE CODE 6003	14 OF 24

APPENDIX E

REPORT

E. D'APPOLONIA ASSOCIATES
CONSULTING ENGINEERS
710 SWISSVALE AVENUE
PITTSBURGH 21, PA.

TELEPHONE
CHURCHILL 2-6530

ELIO D'APPOLONIA
ANTHONY M. DIGIOIA, JR.
RICHARD E. GRAY
JOHN A. HRIBAR
JAMES P. ROMUALDI
JAMES H. SHELLHAMMER

December 12, 1962

Project No. 62-199

Shippensburg Borough Authority
Municipal Building
Shippensburg, Pennsylvania

Attention: Mr. Paul B. Noftsker, Secretary

Preliminary Evaluation of Downstream Drainage
Gunter Valley Dam

Gentlemen:

Pursuant to your request, Messrs. D'Appolonia and Shellhammer of our firm met with you and Mr. Haines of Tracy Engineers, Incorporated, on October 3, 1962, and made a visual inspection of the subject dam. At that time a noticeable discharge of water was emanating from the rock toe at a point just east of the discharge conduit outlet. There was also evidence of previous discharge in the drainage trenches, on the berm along the upstream side of the spillway chute and from the weep holes in the spillway chute. However, we found no apparent evidence that indicated improper construction might be associated with any of the discharges. The magnitude and character (there was no evidence of movement of fines) of the discharge observed at the rock toe indicated no immediate danger to the safety of the dam. However, the magnitude and apparent concentration of the discharge was, in our opinion, of such a nature as to warrant concern about its source, future behavior and corresponding effect on the safety of the dam. We also pointed out then that it might be possible to evaluate these aspects of the observed discharge from existing information and thus avoid an expensive exploratory program. Since the existing discharge presented no immediate danger to the dam, we recommended that this course of action be adopted.

Since our meeting, we have reviewed pertinent available data[1][2] and have discussed ground water and other relevant conditions encountered

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(1) Report to Shippensburg Borough Authority - Gunter Valley Dam Site, by James L. Dyson, May 26, 1959.

(2) Contract Drawings, Shippensburg Borough Authority, Shippensburg, Pennsylvania, Earth Fill Dam and Reservoir and Transmission Lines, Contract No. IV, 1960, Prepared by Glace and Glace, Inc.

during construction with Mr. Barr (resident engineer for the project) of Glace and Glace, Incorporated. The soils and rocks at the dam site are described in substantial detail in the geological report prepared by Mr. Dyson. Mr. Barr has reported that, to the best of his knowledge, the soils and rock formations encountered during construction were substantially as described by Mr. Dyson, both in character and extent. Of particular significance in the present instance is the almost vertical bedding of the rock formations at the site and the position of the Rose Hill sandstone. Beneath the dam the Rose Hill sandstone outcrops in a band about 75 feet wide which extends in a north-south direction almost normal to the axis of the dam. The position of this outcrop is about 70 feet east of the discharge conduit. Of this formation Mr. Dyson remarks:

> "..... There undoubtedly are relatively wide fractures in this rock near the surface, and also at its contact with the adjacent shales. Overburden on this rock is thin, and in places is absent (see Plate 1)."

> "..... These formations, especially the Rose Hill sandstone, would require some grouting to fill fractures. The contact between this unit (IV) and the Rose Hill shale (V) is undoubtedly also a passageway for water and would require sealing."

> "..... About 200 feet upstream from the proposed centerline, where the stream makes a right-angle bend in changing its course from southwest to northwest, there is a steep cut-bank which varies in height from 35 to 75 feet (see Plate 2). The vertical Rose Hill strata exposed in this cliff intersect the center line between holes 23 and 21. There is a strong probability that when this cliff is covered by the reservoir, water will enter the rock fractures and pass through them beneath the dam....."

> "..... Two of these units (II and IV), although hard and dense, do contain some fractures through which water can readily pass....."

Mr. Barr has also reported that during construction of the dam a continuous discharge of ground water emanated from the Rose Hill sandstone outcrop downstream from the dam center line in the vicinity of the present rock toe. The discharge was of a sufficient magnitude to warrant piping it away from the area during construction. Mr. Barr also reported that when this stratum was traversed while constructing the lower end

of the spillway chute, a continuous flow of water discharged from the upstream side of the excavation, and continuous pumping was required to maintain a relatively dry working area. These observations completely substantiate Mr. Dyson's evaluation of the ability of the Rose Hill sandstone to readily conduct water. Furthermore, an examination of the General Plan - Earthfill Dam, Sheet No. 7, of the Contract Drawings in conjunction with Plate 2 of Mr. Dyson's geological report, reveals that the orientation of the Rose Hill sandstone outcrop in the vicinity of the easterly end of the downstream rock toe is immediately east of the point where the concentrated discharge of water was observed during our inspection on October 3, 1962.

Based on the above data and observations it can reasonably be concluded that the source of the discharge noted during our inspection is the Rose Hill sandstone formation which outcrops in the basin. Since the reservoir water level was relatively low at the time of our visit, a relatively dry season of the year, it is also reasonable to conclude that the discharge occurring at that time was about the minimum that may be expected.

Although the source of the observed discharge is rather evident in light of available information, its future behavior and corresponding effect on the condition of the dam cannot be definitely assessed from this information alone. The main point in question is whether the observed flow of water is moving through the sandstone laterally in a zone of relatively shallow depth near the surface of the rock, or is it entering the sandstone upstream in the basin and percolating downward to considerable depth moving laterally downstream, and finally flowing upward and gushing out of the rock surface near the rock toe of the dam. Also of importance is the question of whether or not the flow is affected by fluctuations in water level in the reservoir. Before the effect of discharge on the dam can be adequately evaluated, these two points must be clarified.

It would appear, however, from the limited observations made to date that the flow observed during our inspection is a natural flow of ground water resulting from a gravitational movement of the large quantity of water stored in the sandstone formation and that the contribution to this flow of direct seepage through the dam itself and through the grouted rock formations beneath the dam is small. This flow probably occurs throughout a relatively deep zone of the sandstone, but manifests itself in the observed discharge due to the existence of a path of least resistance in the vicinity of the downstream toe of the dam. Frequently the removal of a small amount of soil overburden is sufficient to completely alter ground water flow and produce springs where previously none

was evident. If the observed discharge is a natural ground water flow it is doubtful, unless a very elaborate pattern of grouting were employed, that such flow could be effectively reduced, or eliminated. Furthermore, if the discharge is a consequence of such ground water movement, it will have no significant effect on the future safety of the dam, and there is no need for undue concern over its existence.

If, however, the discharge in the vicinity of the rock toe on the easterly side of the discharge conduit outlet shows an immediate response to variations in the pool level of the reservoir (especially when the pool level encroaches substantially on the cutbank in the spillway forebay), there would be reason to suspect that there is direct flow laterally from the sandstone outcrops immediately upstream of the dam. If the increase in the flow is substantial, immediate and directly related to the rise in reservoir water level, then further study should be made of the condition and behavior of the flow to ascertain whether or not remedial measures are necessary; and if so, the appropriate type.

To undertake any remedial measures at this time would be entirely unjustified. Likewise, an investigation involving test borings, subsurface exploration, tracer tests and so forth, in our opinion, is not warranted under the present circumstances. It is, however, recommended that the Authority install a system of weirs to measure the flow from various regions in the downstream vicinity of the dam. The system of weirs should be sufficient to define the ground water and surface run-off attributable to various regions along the downstream portion of the dam. Based upon our inspection of the site on October 3, 1962, six weirs located at the following points should adequately serve this purpose: (1) at the downstream limit of the rock toe immediately east of the discharge conduit outlet, (2) at the easterly end of the downstream rock toe, (3) at the westerly end of the downstream rock toe, (4) at the lower end of the berm on the upstream side of the spillway chute, (5) in the old channel of Trout Run between the end of the outlet channel of the discharge conduit and the end of the stilling basin, and (6) in the old channel of Trout Run just downstream from the stilling basin.

Regular periodic observation of the flows over these weirs and their correlation with the corresponding reservoir levels and precipitation, will yield substantial information regarding the behavior of the discharges which are at the moment of somewhat uncertain concern. By installing these weirs and observing the variations in flow with changes in reservoir level and precipitation over a period of several months, sufficient information will be available to permit a more complete evaluation of the nature and type of ground water movement associated with the observed discharges. Based upon an analysis of such data the gravity of the condition can be more definitely established.

If the results of this relatively inexpensive program of measurement and observation indicate that the discharge is of a nature which may eventually endanger the dam, then there would be a much sounder basis for undertaking a more complete investigation and/or possibly remedial measures. If such is the case, the data collected during the recommended program would be extremely valuable in pointing out the type of additional study and/or remedial measures that would be most appropriate and economical as well as effective.

It is recommended that installation of the weirs be accomplished as soon as practicable, and observation of flows and corresponding reservoir levels and precipitation be initiated immediately thereafter. In the meantime, as well as subsequent to installation of the weirs, frequent observation of the discharges should be maintained so that any movement of fines that might develop can be detected as soon as possible. In the event that such a condition should develop (although it appears unlikely), the reservoir level should be lowered immediately and the situation should be given the utmost immediate attention and a course of action adopted consistent with the gravity of the situation.

We would be pleased to meet with you and Mr. Haines to formulate the details of the program outlined above.

Very truly yours,
E. D'Appolonia Associates

James H. Shellhammer

James H. Shellhammer

JHS:ls